MODEL STUDIES

RELATED TO LOADBEARING BRICKWORK

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by

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PRINCIPAL NOTATIONS

H - total height, in

h - storey height, in

l - distance between centroid axes of two walls, in

b - clear distance between two walls, in

A_1, A_2 - cross-sectional areas of walls, sq.in.

A - $A_1 + A_2$

$I_{1xx}, I_{2xx}, I_{1yy}, I_{2yy}$ - moment of inertia of walls, in^4

I_{exx} = $I_{1xx} + I_{2xx}$

I_{eyy} - $I_{1yy} + I_{2yy}$

I_p - moment of inertia of connecting beams, in^4

x - distance measured from top, in

y_{yy}, y_{xx} - horizontal deflection, in.

w - intensity of applied loading, lb/ in.

M_{1x}, M_{2x} - bending moments in two walls, in.lb.

T - total shear force in connecting medium, lb

m_o - bending moment of reduced structure due to wind-loading - in.lb.

m_1, m_2, m_3, m_4, m_5 - bending moment of released structure due to applied shear force in.lb.

m_{xx}, m_{yy} - bending moment on released structure due to unit load, in.lb.

$\Delta_1, \Delta_2, \Delta_3, \Delta_4, \Delta_5$ - horizontal deflection at each storey (Eq. xxiv), in.

δ - Total deflection at roof level, in.

δ_2 - " " " 2nd floor level, in.

V_1, V_2, V_3, V_4, V_5 - horizontal loading at each slab level, lb.

G_1, G_2, G_3, G_4, G - modulus of rigidity of each storey, lb/in^2

CHAPTER I

General Introduction

1.1. History:- The structural and functional use of brick wall carries us back to the dawn of the civilization; near the ancient temple and palaces[22,35] at Tepe Gawra in Mesopotamia built in 4000 B.C.; near the ruins of brick buildings of Indus Valley cities[22,67] (2,500 B.C.), near the sophisticated tombs built by Egyptian Master Builders[22,36] (3,000 B.C.) and later to the multi-storey shopping centres of Romans[36]. In 1891 A.D., brick masonry designed by traditional practice and rule of thumb, reached its peak when the 16-storey Monadnock[22,27,33,67] building, the highest in the world was completed in Chicago with brick bearing walls 6 ft. thick at the base. A plaque was put which commences "the final triumph of traditional masonry construction"; this virtually sealed the fate of load-bearing brickwork. Since then it suffered a rapid decline and its use was fairly limited on a small scale for domestic buildings. "Necessity is the Mother of invention"; the economic necessity and bricklayers strike forced William Le Baron Jenny (1893 A.D.) to develop the structural[27,63,65] frame as an alternative to the brickwork for the Home Insurance Building in Chicago. Really, it was irony of fate that the designer of Monodnock Building, John W. Root[27] should also be amongst the first to use the frame. The development of structural frame replaced the structural use of brickwork in multi-storey building and limited its use simply as curtain wall/

wall to support its own weight. Increased use of metal curtain wall by the end of the second world war and finally of glass by the second half of the 1950's culminated in the complete disappearance of brick from numerous modern multi-storey buildings.

In comparison with brickwork buildings the all glass building is devoid of textural warmth and colour and is most unsatisfactory from the environmental point of view. During this decade Architect and Engineer again turned to the use of brick for multi-storey buildings, utilising its structural strength and aesthetic qualities. The brick building is not longer treated as a piece of traditional craftmanship but analysed and designed according to similar techniques as have been used previously for steel, reinforced concrete and timber. Switzerland, having no governmental bye-laws or Code and also have no indegenous steel industry became the pioneer in this field. The first scientific investigation[27,63,33] on the strength of brickwork was conducted on 1,600 wall specimen at EMPA (Swiss Federal Materials Testing and Research Institution), which revolutionised the use of brickwork and laid the foundation for the design and construction of three 13 storey apartment buildings (Plate 1·1) built in Basle from 1951-53. Adhering to this design technique in 1957, the Swiss built an 18 storey high slab block (Plate 1·2) the tallest load-bearing brick building[27,62] in the world. Since then multi-storey buildings supported on relatively thin brick masonry walls have sprung up all over the world as a result of flexibility, economy and speed afforded by this means of construction.

In this country, between 1926-1934 intensive research was carried out, largely at Building Research Station[19] on square brickwork piers which formed the basis of 1948 code. Though a substantial amount of data was available on the/

the strength of brickwork by 1953, it was not harnassed to any appreciable extent till 1960. However, the construction of a 12 storey flats (Plate 1·3) at Birmingham[27,63] and the Swiss experience had great impact, which resulted in the issue of revised 1964 Code paving the way for more extensive use of load-bearing brickwork building in this country.

In Denmark, a large number of brick pier tests were done under late Professor Svenson[63] in the early 1930's. His work emphasised that the modulus of elasticity of the individual brick unit has profound influence on the compressive strength of masonry. According to him the variation in the modulus of elasticity between brick units gives rise to internal eccentricities or localised stress concentrations resulting in a non-uniform pattern of failure over a given cross section. Due to the above research, Denmark built 9 and 10 storey apartments in Copenhagen (as early as 1943) in 9" load-bearing crosswalls stiffened by a central spine. In 1963, a 16 storey building, supported on 14" thick walls (Plate 1·4), the tallest load-bearing[27,63] brickwork in Denmark was completed and strain gauges of gauge length equal to the wall height were fixed at thirty-five selected points in ground floor mid height and top storey. The report of the observations is still awaited and this may throw some light on the behaviour of structure due to wind loading. Like Swiss, Danish building economy is also suited for the development of load-bearing brickwork as they also have no steel industry.

About the same time, Professor Onishchik[43] in U.S.S.R. conducted lot of tests on brick piers and walls and suggested emperical formulae based on brick strength to predict the compressive strength of brickwork. Most of the results of his investigations were published in book form in 1937 and this record still influences considerably the Soviet thinking.

Unfortunately,/

1 - Plate 1.4

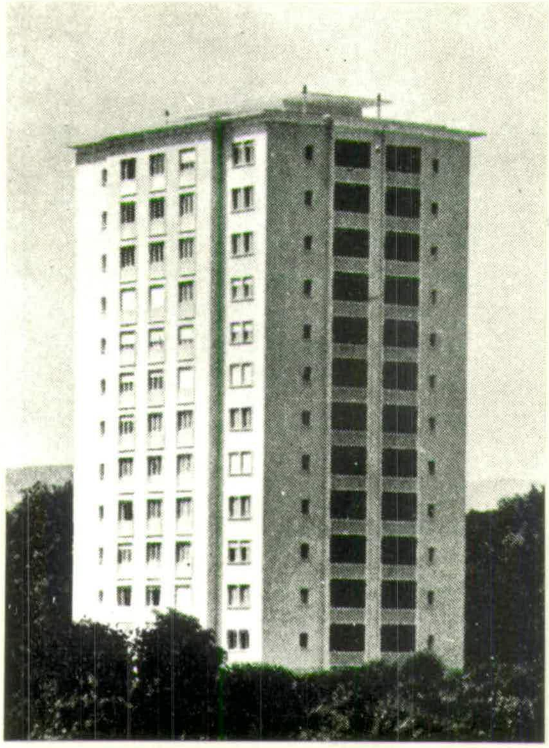


Plate 1.1 - Flat in Basle, Switzerland

Plate 1.2 - 18-storey block in Schwamendingen, Switzerland.

Plate 1.3 - Flat in Birmingham, England.

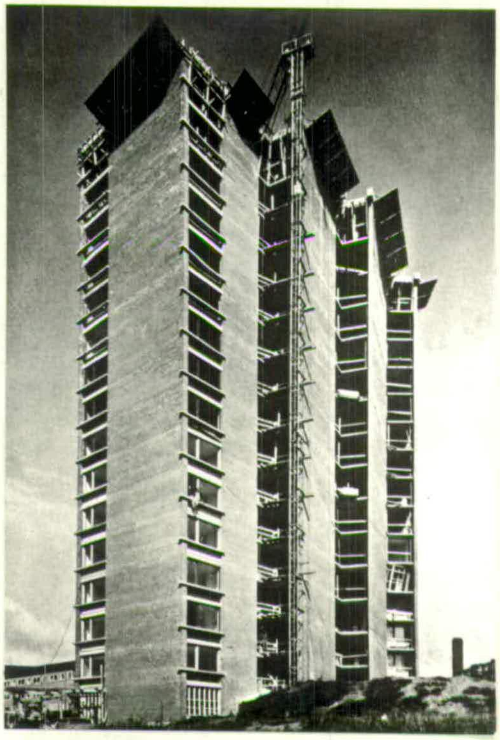


Plate 1.4 - 16-storey Building in Aarhus, Denmark.

Unfortunately, neither Svenson's nor Onischik's work is available in English and did not get wide publicity.

In U.S.A. (1882-1912) a series of tests on brick masonry piers were done at Water-town Arsenal and in different institutions[37]. From these tests the influence of mortar composition and unit strength on brickwork were recognised. In 1915 a series of tests on brick piers and walls similar to those tested in this country were done at the National Bureau of Standards and ultimate strength of brickwork for different strength of brick was calculated statistically by fitting a linear equation to the data.

Nothing very exciting in brickwork happened in U.S.A. until September 1950 when Robert L. Davison[21] put forward his new imaginative concept of crosswall type of construction for multi-storey buildings. In this system, the brick wall in addition to the function of carrying the vertical load, carries the horizontal load as well. The horizontal load caused by wind from any direction is distributed by the rigid floor and roof slab to the cross-walls and shearwalls and thus full advantage of racking resistance of brick wall is taken. The Structural Clay Products Research Foundation of America, with the collaboration of Illinois Institute of Technology, intended developing this system as an improvement to residential design and construction and hoped to conduct certain tests but nothing has been published to date. However, four to nine storeys high blocks in 12" load-bearing brickwork with few similar blocks in 16" at Pennley Park and a 17 storey block in 11" thick R.B. at Park Mayfair East are a few examples of Multi-storey[23] buildings in masonry. In America/

America till now the design of masonry walls is not controlled by limiting stresses in the walls, but by the minimum thickness designated in the various building codes of the United States, and this was followed for the design of above buildings.

Small scale tests have been carried out on single leaf prisms for compressive strength, flexural strength and diagonal tension on 31 different[42,61] kinds of brick produced in the U.S.A. by S.C.P.R.F. from early 1963. The aim was to recommend the simple test for design and field control. Some tests on full scale walls were also carried out and it is hoped that in future a 25 storey high building would be possible in 8" brick bearing walls[23].

The Swiss have put considerable emphasis on the compressive strength of the brickwork and almost neglected the shear strength. The advantage of masonry as primary lateral load resisting element has been recognised during the second world war and after the completion of the Empire State Building[39]. This aspect of brickwork masonry has been only investigated on fairly small scale couplets and individual walls by the research workers in U.K., U.S.S.R., U.S.A. for drawing up of a code.

1.2. Modern Trend of Research

It is realised now that intensive and sophisticated research must be done to study the behaviour of complete masonry structures to exploit the material fully and economically, so that it may co-exist beside the structural steel and reinforced concrete in multi-storey buildings of today. Earlier research failed in this respect, as it has been carried out on isolated walls or piers. The walls were tested between knife edges, which did/

did not simulate the actual condition in a building. Some work has been carried out on interaction of wall panels and floor slab simulating the actual condition in building to determine the load-carrying capacity of wall (Sahlin[51] - 1959, Prasan[48] - 1963, Bradshaw[8] - 1966). The tests results of Prasan[47,48] and Bradshaw[8] suggest that C.P.111 - 1964 is still conservative for the allowable stress in brickwork. Murthy[39] (1964) developed model technique to examine the strength of storey height brick walls and shear walls with particular reference to the effect of precompression. This finally led to the study of stiffening effect of shear-wall in a three storey, three bay, assembly of crosswalls.

The ultimate strength of the structure was very high compared to the strength of storey high shear wall and it was thought not to be very conclusive at that time.

1.3. Present Scope of Work:-

While the bulk of the walls tested were on single leaf, in this work the effect of brickwork bond on the strength of 9" wall has been examined, under conditions simulating these in a building.

The influence of the moisture content of the brick before laying and the load placed on the couplet during curing on bond tension and shear has also been studied. These tests were conducted to study the effect of pre-compression on the shear strength of couplets to collect data for further investigation of the cross-wall structure.

Previously, the shear wall was treated as isolated element and hence shear test was also conducted on isolated wall with concentrated restraining load on top rather than realistic uniform load from slab. In actual fact, the/

In actual fact the crosswall acts integrally with the floor, shear wall and roof and form a highly complex three dimensional structure system. Unless the composite behaviour is fully understood the strength cannot be fully and economically exploited. An attempt has been made to study the ultimate behaviour of single storey crosswall structures containing openings and subjected to precompression. Finally, an investigation has been carried out to study the interaction between slab, cross and shear walls and the ultimate load behaviour of a complete 5-storey crosswall structure with openings, this represents a unique undertaking in the field of study encompassing actual brick structures.

Efforts were also made to correlate the model test results with the full scale tests obtained from B.R.S. and elsewhere.

1.4. Experimental Technique:- The analytical solution of shear wall is based on linear elastic behaviour, whereas the behaviour of brick structure is neither linear nor elastic. Further, theoretical solution based on these assumptions becomes null and void for large deformation and ultimate behaviour near failure. In the case of brick shear wall a small increase in vertical load may change the behaviour of structure, which further complicates the problem. As in this case theoretical solution fails, the experimental approach became the obvious choice. However, it is very expensive and time consuming to study the behaviour of complete structure at full scale. Test on reduced scale models to represent the characteristic properties, appear to be the answer to the problem. Hence, in the present work 1/6th scale model brick has been used to study the behaviour of the load bearing brickwork structure, the validity of the technique has already been established[39,40,4].

CHAPTER 2

A review of previous work on Model Analysis for structure in Brick Masonry.

Model testing[14] is not a novel idea. In the middle ages models were made to demonstrate the pattern of forces, but they were not meant to be used as loading tests. Though model studies have been done frequently for the analysis and design of structures, Benjamin and Williams[5] (1952 - 56) appear to be the first to use this technique to investigate the behaviour of brick masonry shear walls under lateral load. The shear tests were carried out on large size and scale model walls, with or without bounding frames of steel or concrete. The first problem encountered was to find out whether a scale effect was involved. A series of tests on plain infilled brick panels of sizes varying from 0.34 full size to full size were performed to s tudy the scale effect. A standard joint thickness was used regardless of the modal scale and wall thickness was changed by different orientation of the brick used. Further model tests were also performed using the cut bricks and the results were compared with those for the panels in which the bricks were not to the model scale. The load deflection curves obtained from the model experiments and adjusted for the scale factor, were compared with those of the full-scale. There was a wide scatter of experimental results but the ultimate and first crack load varied much as in scaled and full size walls and this led to the conclusion that the scale effect is insignificant.

Although the technique opened an avenue for the investigation of brickwork structures, it had its limitations, The scale model only approximately/

approximately represented the full scale panel which was 8 ft. high and 12 ft. long. The choice was limited to 0·48 and 0·3 of full size for studying the behaviour of 8 5/8" thick wall with brick of 8 5/8" x 4 1/8" x 2 5/8" as the width and thickness of bricks represented the length respectively. This technique will work for studying the behaviour of walls in shear, but may not work under different conditions of loading. It will certainly not work for the investigation of the ultimate behaviour of wall panels in compression, where the thickness of the mortar joint exerts a considerable influence.

Speer[60] (1953-54) carried out photoelastic investigations to find out the stress distribution in compression, using an elastic and a solid bedding of the bricks. The model bricks were made of Eilenberg Synthetic resin and the elastic and solid bedding of the bricks were attained by using strips of linoleum or cement, made of fine sand and Dusan glue, respectively. These tests indicate the existence of tensile or compressive stresses in the horizontal direction due to the deformation of the bricks. These stresses were maximum in the middle third part of the brick and decreased rapidly towards the end. In one set of tests, where bricks of different height with elastic bedding were used, bending stresses were induced.

It may be possible by photoelastic tests to examine the stress distribution pattern in brickwork provided a proper choice of material is made for the "mortar" and the "brick". This will serve only to give an insight to the problem on qualitative basis.

Vogt[66] (1957-58) carried out some preliminary tests to provide a basis for model tests on brickwork in compression. The model bricks used were

60 m.m./

60 m.m. x 29 m.m. x 20 m.m. Tests were conducted on four groups of eight pillars measuring 6 cm. x 6 cm. x 30 cm. Mortars of different strengths were used for each pair of pillars forming a group. In group four, the joint between bricks consisted of cardboard strips instead of mortar. In the tests (1 to 3) very small increase in brick strength was noticed, though the mortar strength was increased to 11.3 times. Very high ultimate stress was obtained in case of cardboard joint and it was concluded that the tensile strength of material is the decisive factor. There was wide scatter of experimental results, mainly due to the dimensional inaccuracy of model bricks. No attempt was made, however, to correlate the results with full scale tests.

Murthy[39] (1964) developed the model technique of testing brickwork structure to a great extent. A large number of tests were done in 1/3rd and 1/6th scale brick piers to reproduce the tests previously done on full scale at B.R.S., taking into account the effect of mortar strength, slenderness ratio and eccentricity. The joint thickness was also sealed down and 1" cubes were used to determine the mortar strength[39,40]. The results on eccentrically loaded piers are possibly better than these on mortar strength. Model test results were plotted in non-dimensional quantity as shown in Fig. 2.1 to obtain a comparison with full scale. Though there was some scatter of experimental results - generally it was found quite consistent with the full scale test. The failure characteristic of model and full scale were the same. Tests carried out on storey-height $4\frac{1}{2}$" wall by Prasan[48] (1963) were also repeated at 1/6th scale.

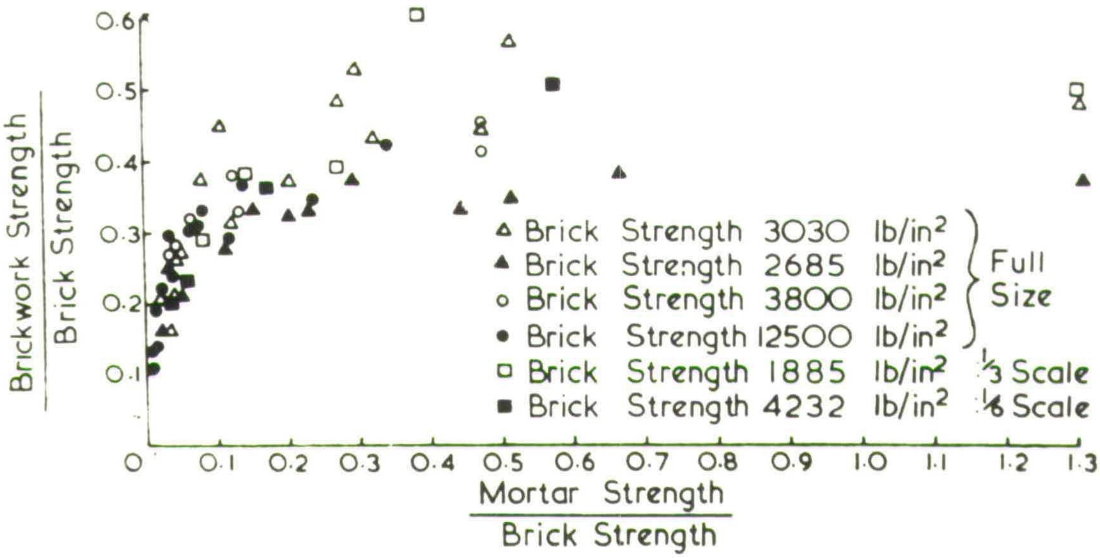


FIGURE 2.1 – Test results: brickwork strength v. mortar strength

The result of model were adjusted according to the relationship established by Davey and Thomas[19] and good agreement was found. These tests demonstrated without doubt that the model technique could be adopted for the investigation of brickwork structure. However, no attempt was made to compare the result of shear test with full scale test done elsewhere.

CHAPTER 3

THE EFFECT OF BRICKWORK BOND ON THE LOAD-BEARING CAPACITY OF MODEL BRICK WALLS

3.1. INTRODUCTION

The purpose of the tests was to study the effects of brickwork bond on the load-bearing capacity of walls. It has already been established by MURTHY and HENDRY[40] that the strength of full-scale brickwork for a given strength of brick and mortar, may be reproduced by means of model tests - provided that 1-in. mortar cubes are used for the determination of mortar strength. It was therefore decided to use one-sixth-scale model brick to investigate the effects of bond of different types. The bonds used were:

English - Alternate header and stretcher courses.

Flemish - Header and stretcher alternately in same courses.

Garden - One course of header to three courses of stretcher.

Header - All header courses.

Stretcher - All stretcher courses with and without ties.

Two walls were built of each bond except for the English type, for which three walls were constructed.

The wall panels were tested between reinforced concrete slabs to simulate the end conditions of a wall in a building.

3.2 MATERIALS

3.2.1 BRICKS

One-sixth-scale model bricks with an average crushing strength of 4227 lb/in^2 were used for the tests. The measurements shown in Table (P.22) 3.1 give an indication of the dimensional accuracy of the model bricks used in/

in these tests. The physical properties of the bricks are given in (P.23) Table 3.2. The axial strength of the model bricks is approximately 40 percent of the transverse strength, which is in agreement with the findings of the National Bureau of Standards on the full-scale tests, reported by CARAVATY and PLUMMER[13]. The typical failure of the bricks in axial tension is shown in Plate 3.1.

3.2.2 SAND

The sand used was dry Leighton Buzzard No.19, the grading of which is shown in Tables 3.3 (P.23) and Fig. 3.1.

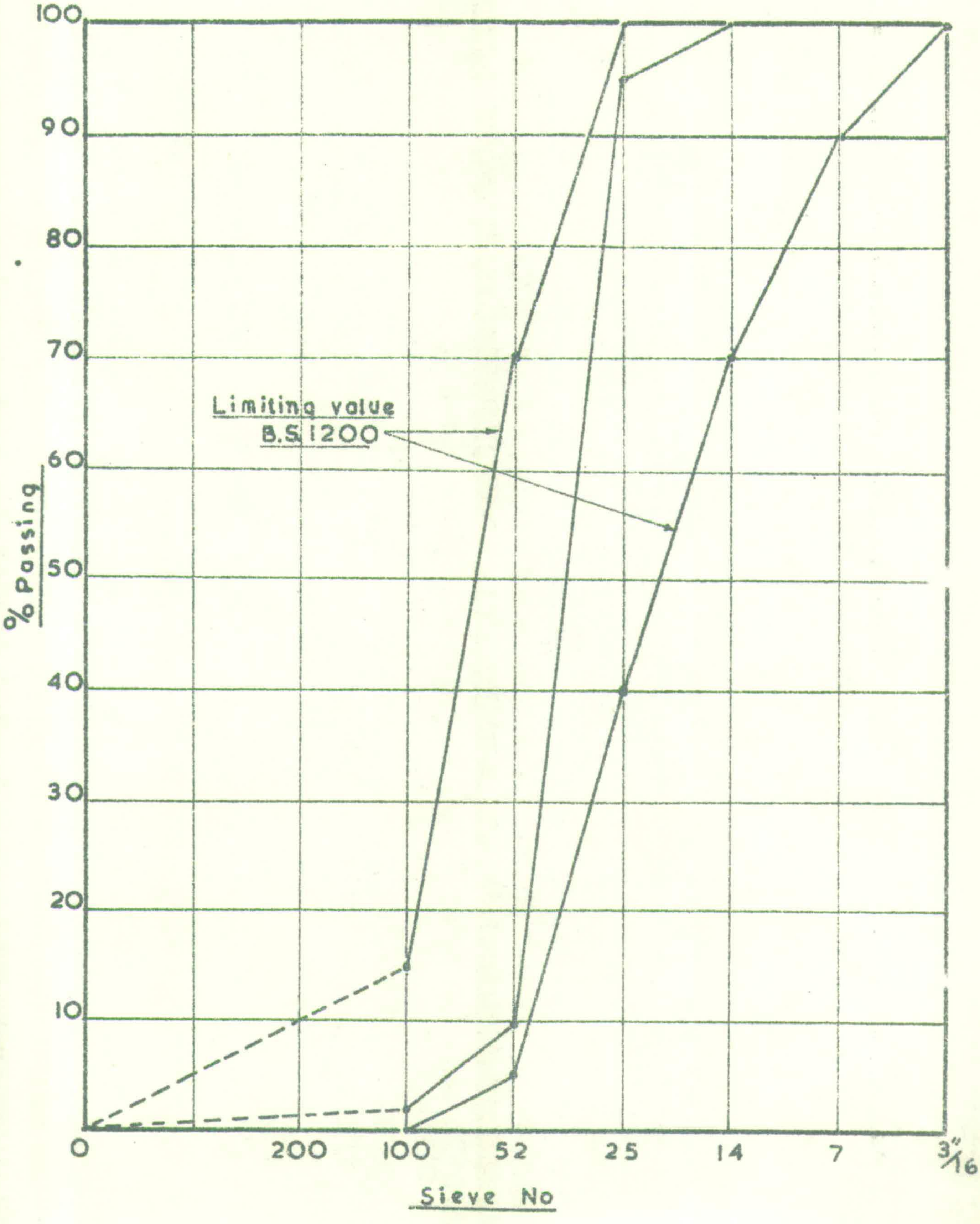
3.2.3 CEMENT

A rapid hardening cement, 'Ferrocrete', was used for all the tests. The average compressive strength of 2.78 in. cement mortar cubes was 4928 lb/in^2 at the age of 7 days against the minimum of 4000 lb/in^2 recommended by the British Standards Institution[9].

3.2.4 MORTAR

The mortar mix was made up by weight to the proportion of 1:4 cement/sand. This was obtained by converting a 1:3 volume mix. The average crushing strength of the mortar cubes is given in Table 3.4. (P.24) The cubes in each case were crushed the same day as the corresponding wall tests.

Fig. 3.1. Grading curve of Leighton Buzzard sand.



The mortar joining the wall to top and bottom slabs was a 1:1 cement/sand volume mix. The average crushing strength of these 1-in. cubes for joining of all the various walls on the third day, the day of testing the wall, was 1120 lb/in^2.

Trial mixes of 1:3 cement mortar with different water: cement ratios were tested by dropping a 1-in. metal ball weighing 65 gms. from a height of 10 in. The depth of penetration was measured, and, when pentration was 15 m.m., the corresponding value of water cement ratio (0·91) was found satisfactory for the brickwork and adopted for all the tests presented in this thesis.

3.2.5. CURING

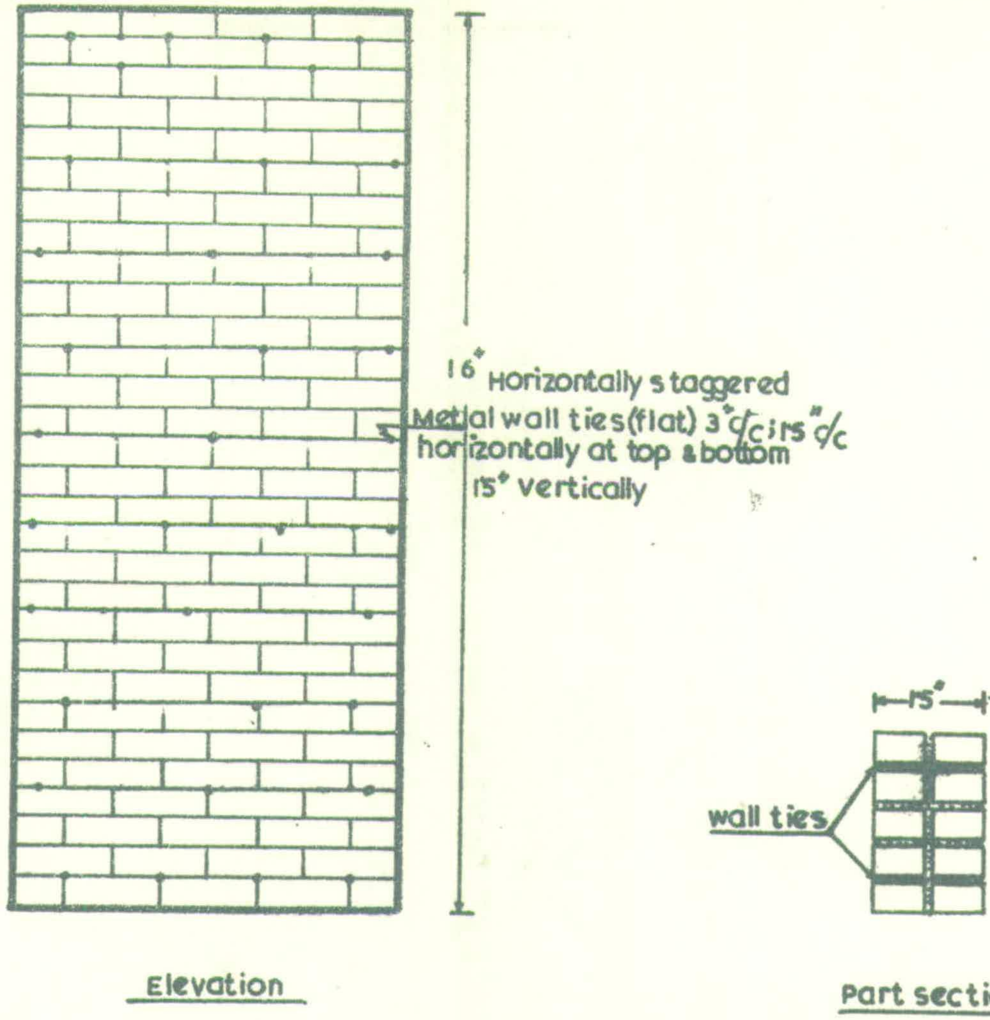
The mortar cubes were initially kept at 99 percent humidity for 24 hours after which they were removed from the mould and stored along with the test walls under damp sacking for 3 days. They were then cured in air in the laboratory. All specimens were tested after 28 days.

3.2.6. WALL-TIES

Metal ties, 1.2 x 0.125 x 0.02 in., were used for 9-in. stretcher bond with ties. The shape and location of the ties are shown in Fig.3.2.

3.3. METHOD OF BUILDING THE WALL

It was too difficult to construct the model walls accurately in situ, hence a jig of the required dimensions was made. Lines were drawn on the plywood backing of the jig to indicate the correct height of the individual courses of the brickwork. The walls were built vertically against the plywood backing of the jig as shown in Plate No.3.2.



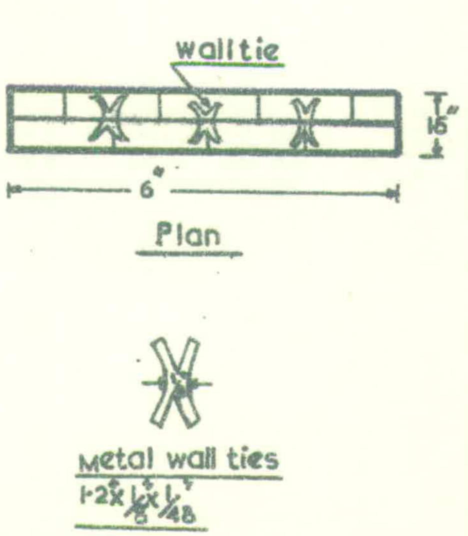


Fig.3.2 - Showing the position of the wall ties in the stretcher bond.

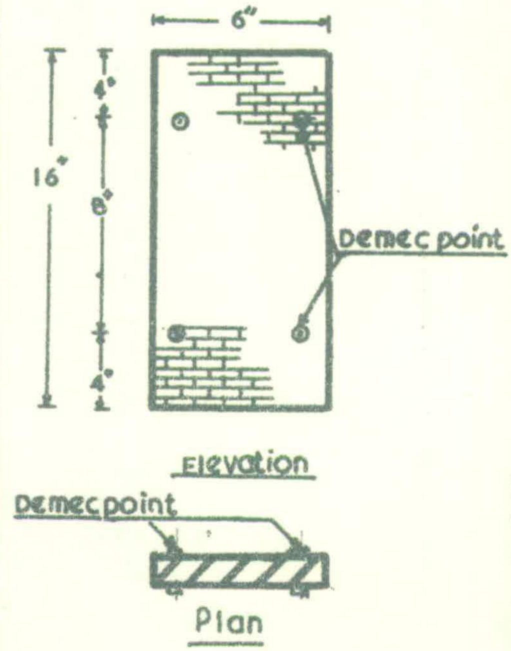


Fig.3.3 - Showing the position of demec points.

<u>3 - Plate 3.1 - 3.2</u>

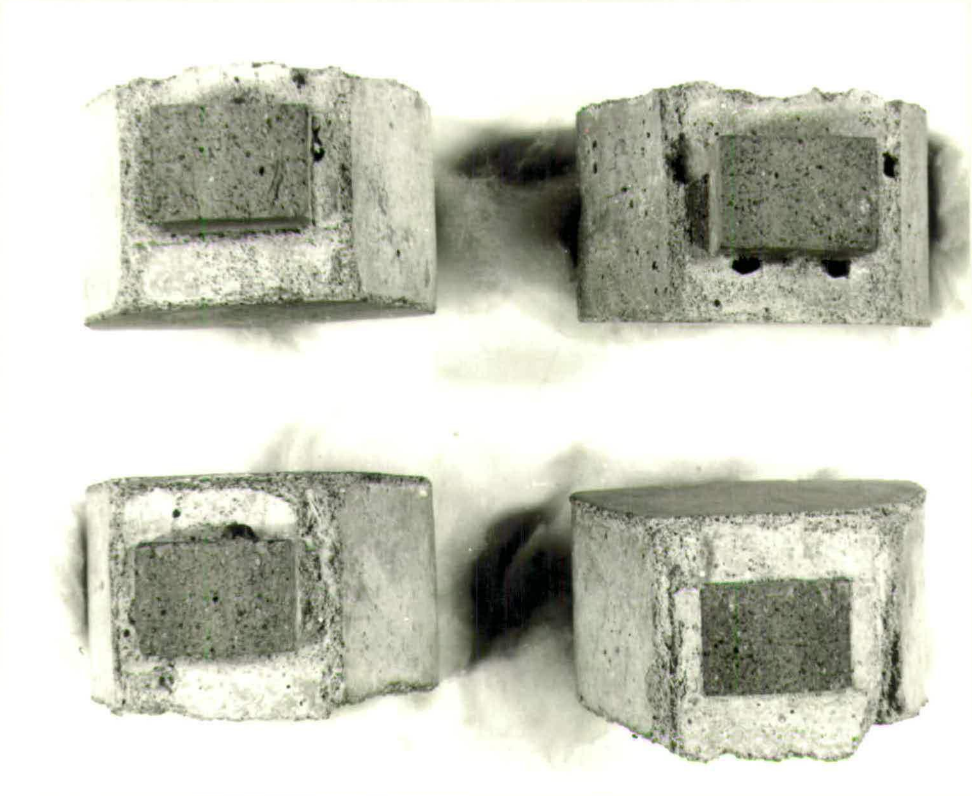


Plate 3.1 - Typical failure of brick in axial tension.

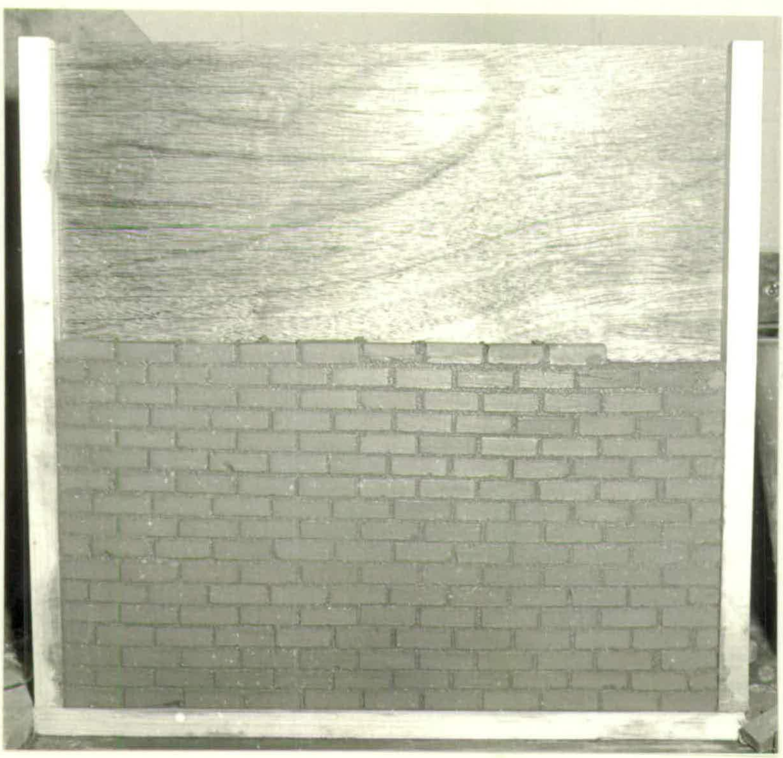


Plate 3.2 - Method of Construction of wall.

3.4. TESTING EQUIPMENT

A 'Demec' gauge of 8-in. nominal length was used for measuring the strain in the walls. The strains were measured on both faces of the walls as shown in Fig.3.3.

A proving ring of 10-ton capacity was used to measure the load. It was calibrated in an Avery Universal testing machine. The calibration curve is shown in Fig. 3.4.

The load was applied by means of two 6-ton hydraulic jacks. The test arrangements are shown in Plates 3.3 and 3.4.

3.5 WALL TESTS

The test walls were all 16-in. high, 6-in. wide and 1.5-in. thick. All the vertical joints and bed joints were completely filled and flushed with mortar. The central through joint in the stretcher bond was also completely filled with mortar.

The wall specimens were tested between top and bottom slabs as shown in Plate 3.3. The slabs were 32-in. long, 6-in. wide and made from 1:1:2 concrete. The maximum size of the aggregate used was $^{3}/_{16}$-in. The top and bottom slabs were 1.5-in. and 0.75-in. thick respectively, both reinforced with 1 percent mild steel reinforcement. The average compressive strength of the 4-in. concrete cubes after 7 days was 3990 lb/in^2.

In all the tests the walls were placed on the centres of slabs which in turn were supported at their ends on the angles of the test frame. The bottom slab was also supported in the centre of four courses of brickwork 6-in. wide and 1.5-in. thick. On the top of the slab directly over the wall/

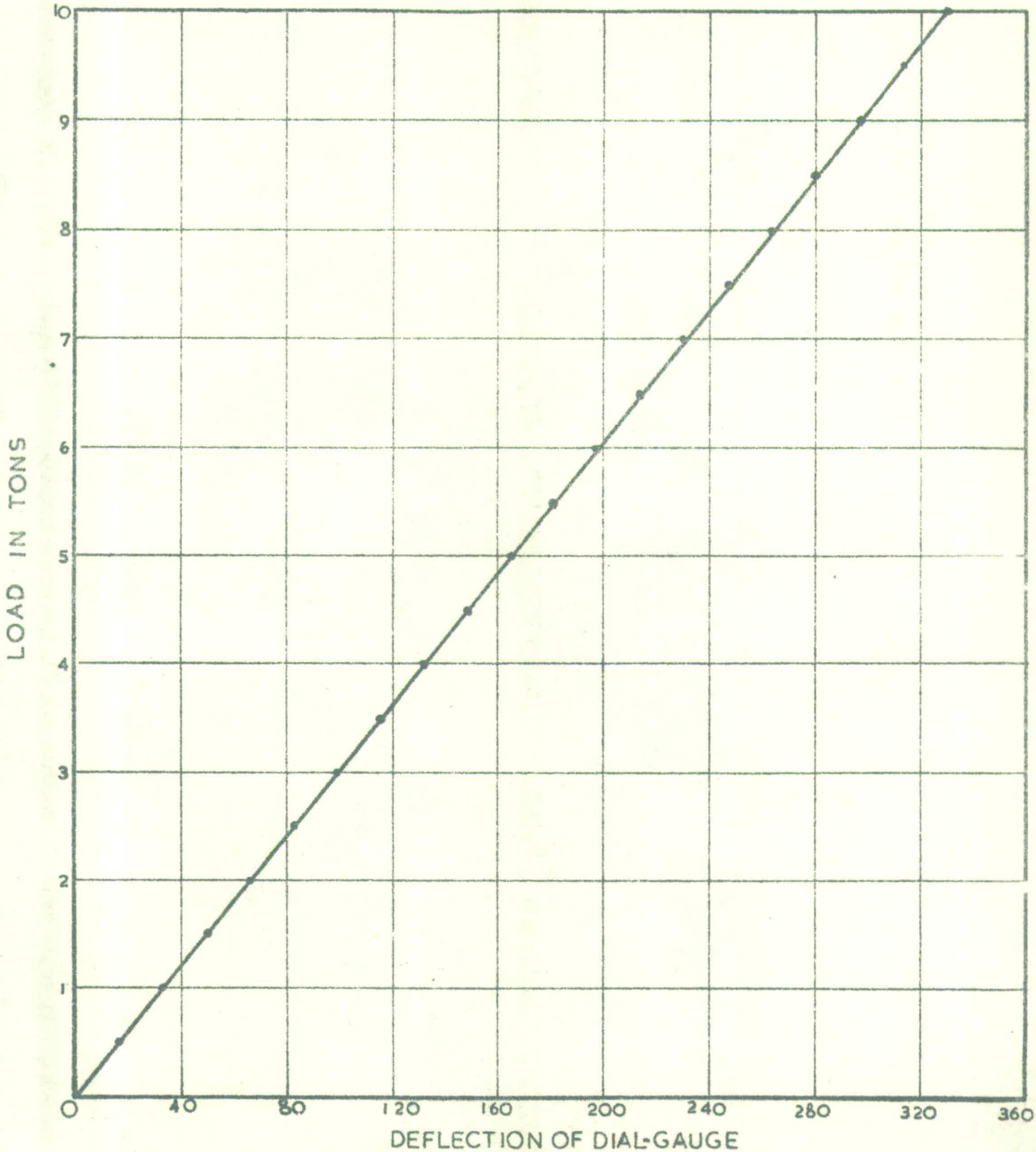


Fig.3.4

Calibration Curve

3 - Plate 3.3 - 3.4

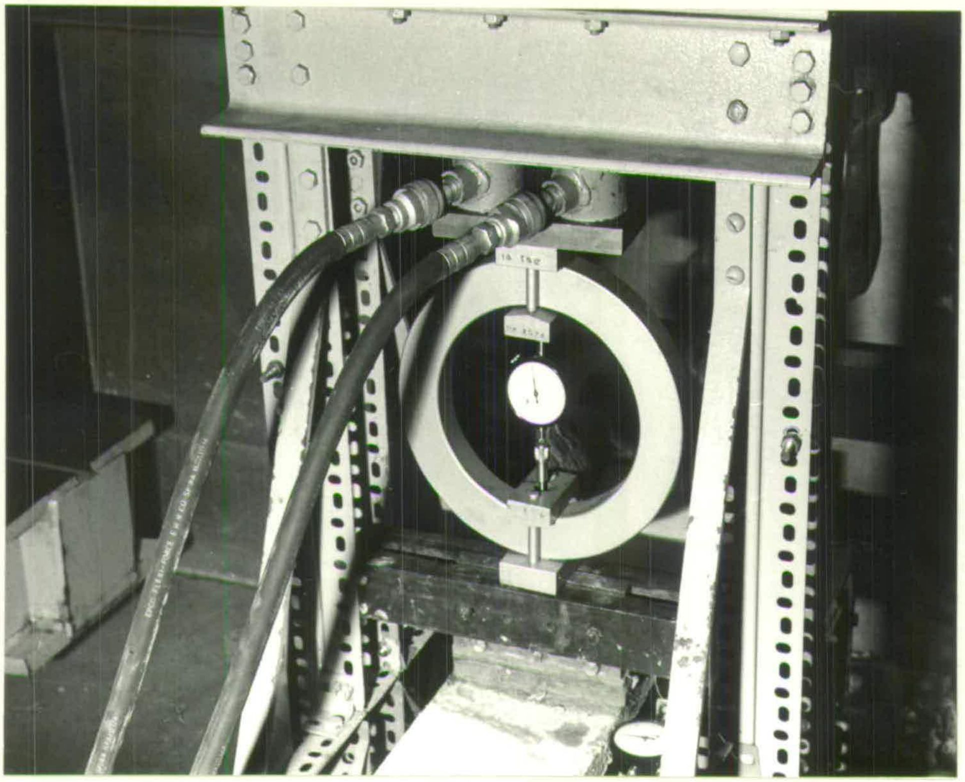


Plate 3.3 - Test arrangement for measuring the load.

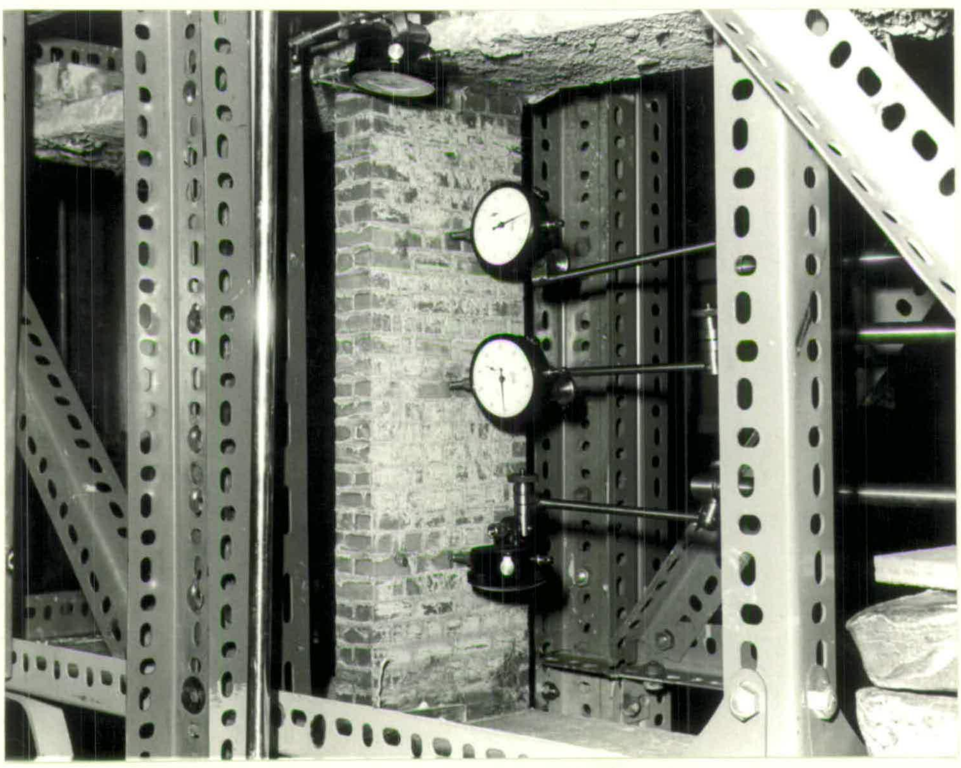


Plate 3.4 - Dial guages for measuring the lateral deflection.

wall one course of brickwork 6-in. wide and 1.5-in. thick was laid. A 1/8-in. thick sheet of plywood was embedded in mortar on the top course of brickwork to distribute the load evenly from the loading beam. The small gaps between the slabs and angles were fully packed with cement mortar.

Each wall was tested to destruction under axial loading. Initial failures were observed in the thickness of the walls in all types of bond with the exception of header and stretcher bond with ties, where the crack first appeared on the face. Failure occurred quite suddenly and in most cases explosively, and was characterized by the appearance of a vertical crack running from top to bottom of the wall, either in the thickness or on the face of the wall.

In the case of the English-bond walls nos. 1 and 3 (Table 3.4) it was not possible to reach the failure load. At 10 tons, the recommended limit of the proving ring, the walls did not show any evidence of failure and the load was increased to approximately 11 tons. Wall No.1 did not fail. Wall No.3 was unloaded and reloaded several times and at the fourth attempt it failed when the load was allowed to remain for 20 min.

A summary of the test results is given in Table 3.4 (P.2[illegible]). Plates 3.5 to 3.8 show the typical failure patterns of the walls. The relationship between compressive stress and strain is shown in Figs. 3.5 to 3.16. The load deflection curve is shown in Fig. 3.17.

3.6. DISCUSSION OF TEST RESULTS

From the results of the tests it appears that the load-carrying capacity of brickwork is not affected significantly by the brick bonds investigated./

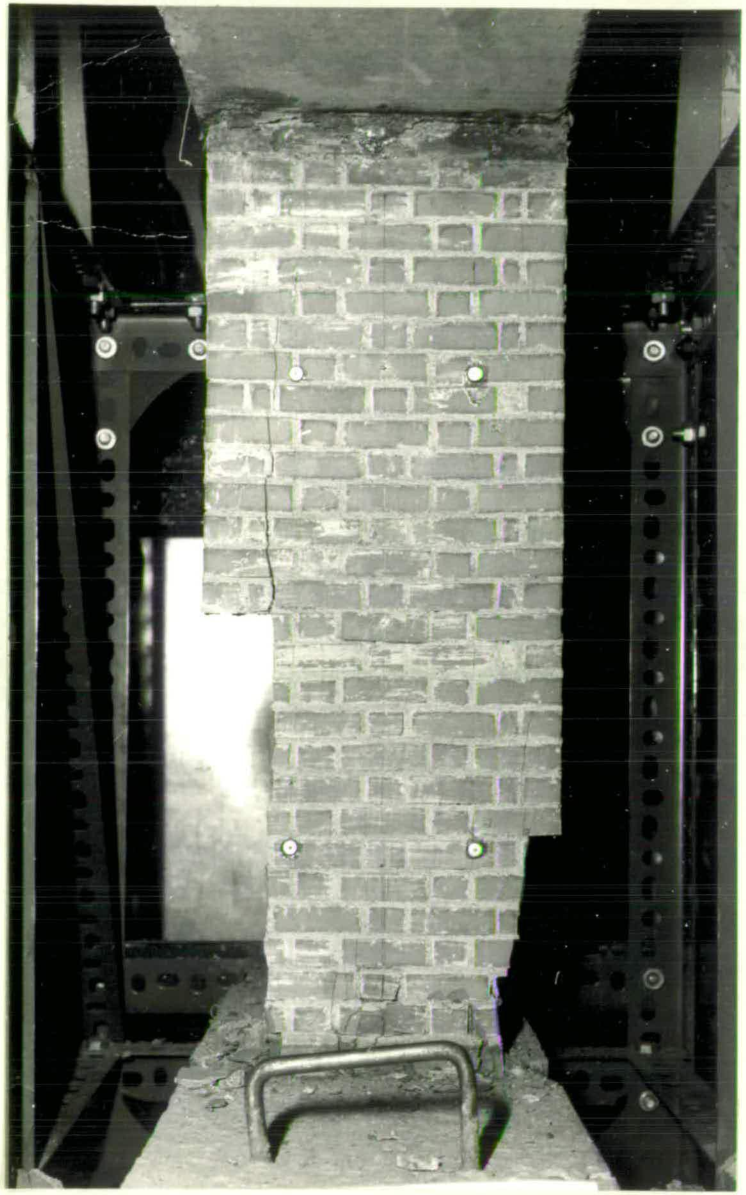


Plate 3.5 - Typical failure by vertical splitting of wall (Flemish bond)

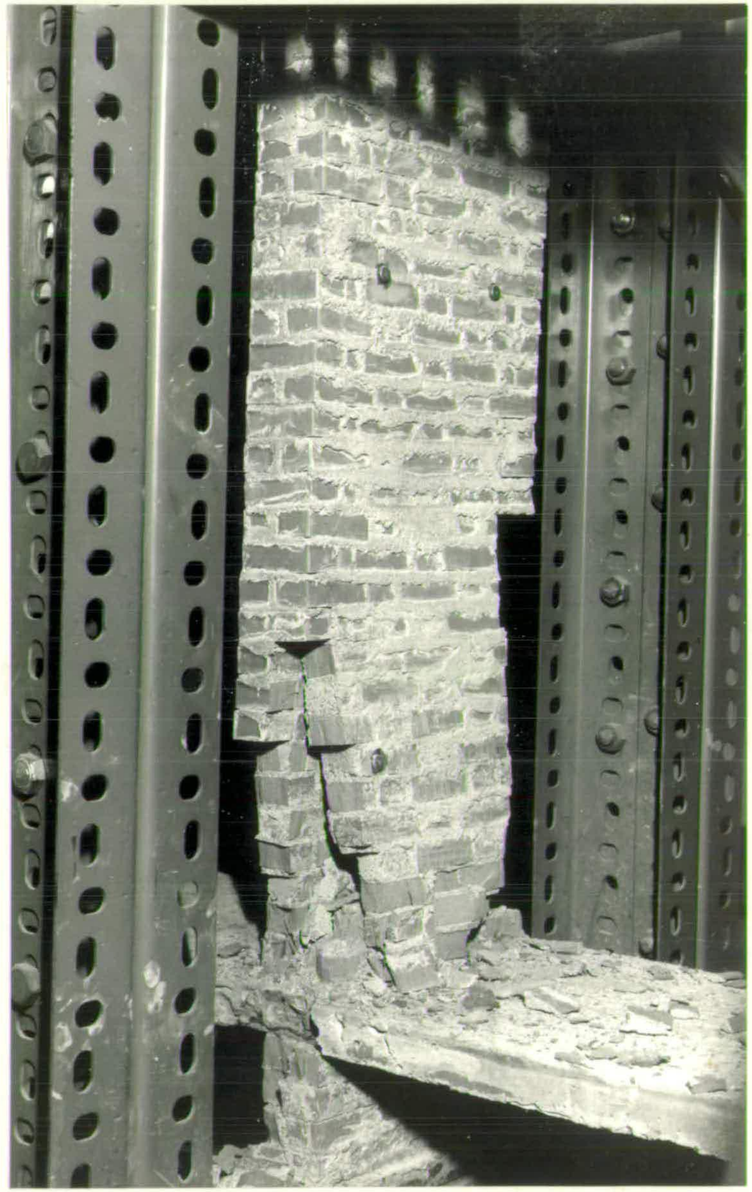


Plate 3.6 - Typical failure by splitting in the thickness of wall (Flemish bond)

3 - Plate 3.7 - 3.8

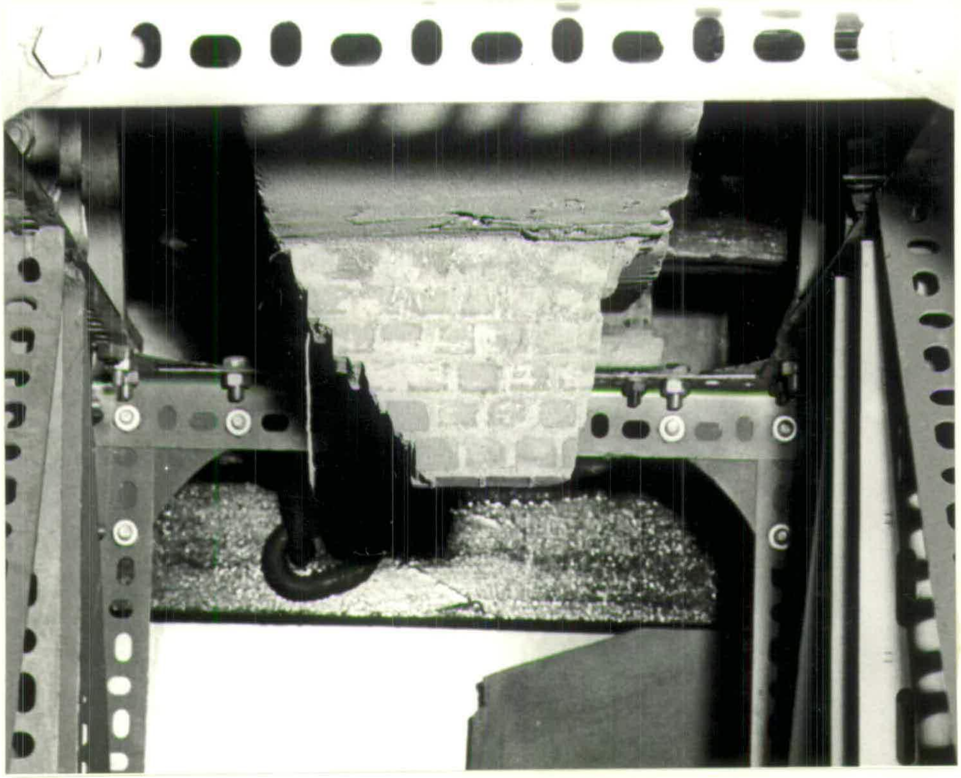


Plate 3.7 - Typical failure of header bond.

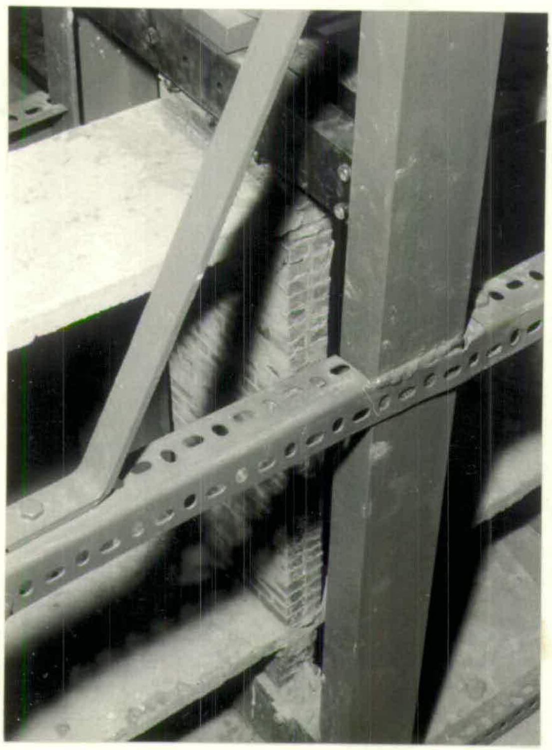


Plate 3.8 - spalling in the top of the wall (stretcher bond with ties)

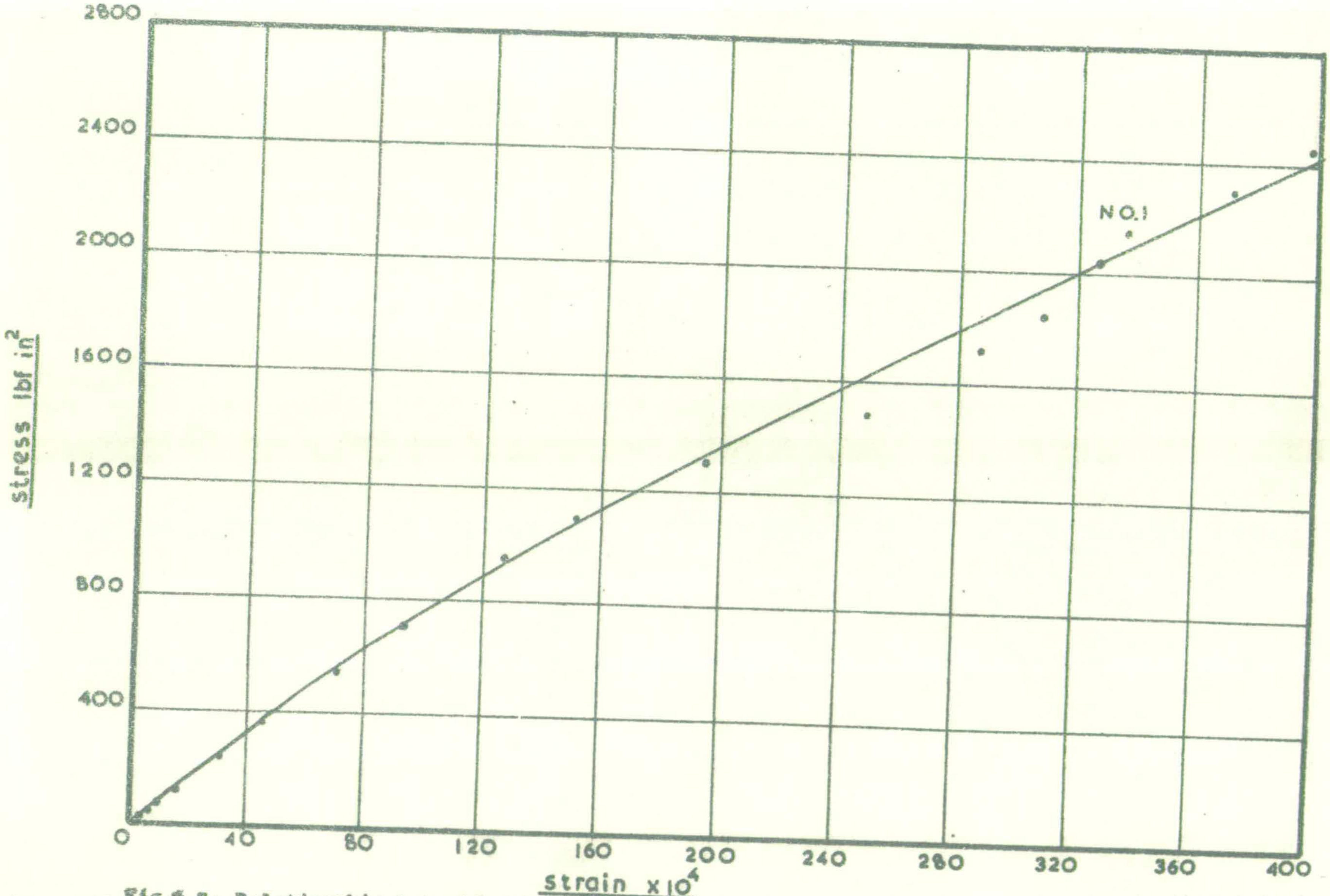


Fig.5.5- Relationship between stress and strain (English bond).

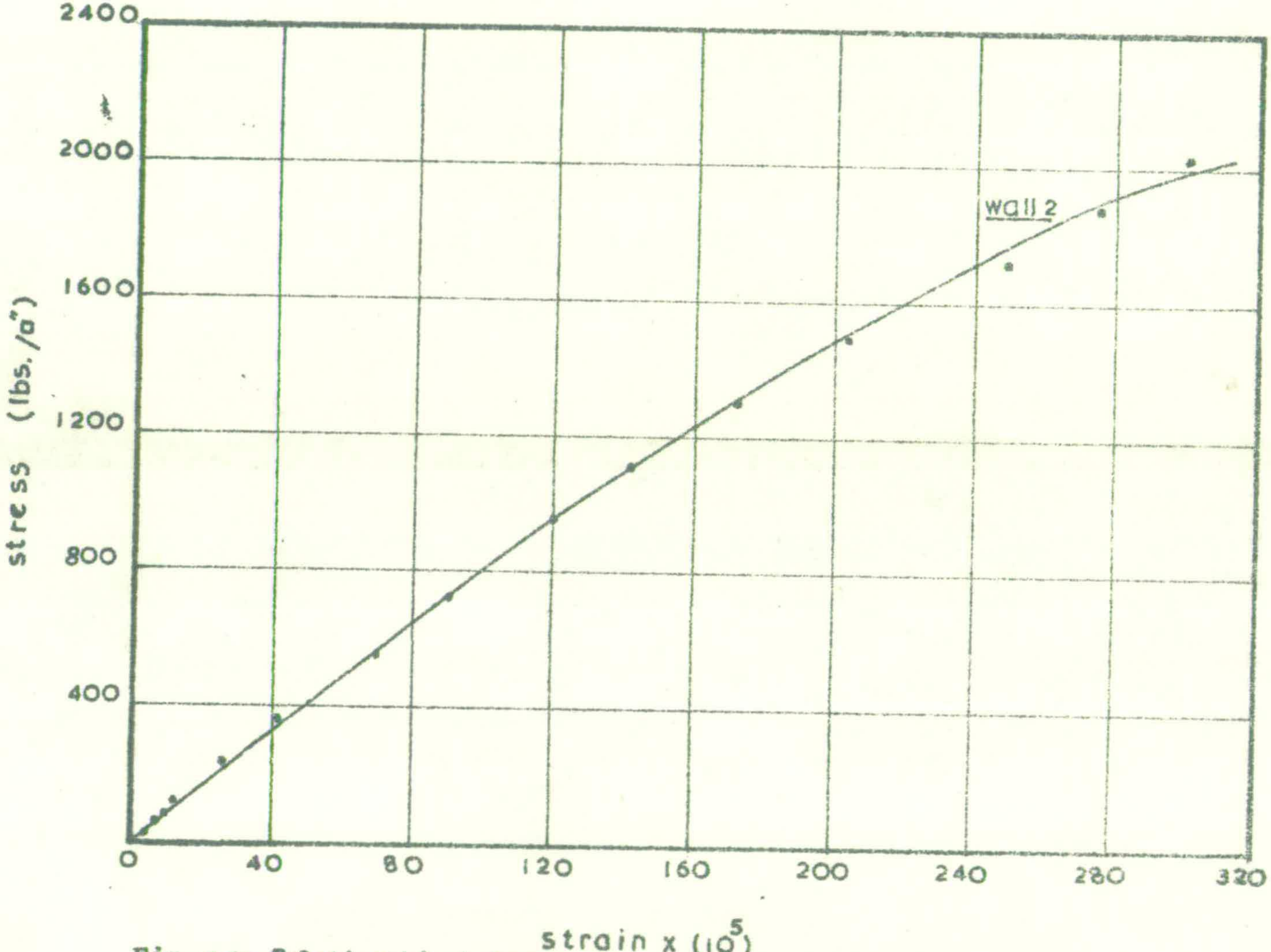


Fig.-3.6- Relationship between stress and strain (English bond).

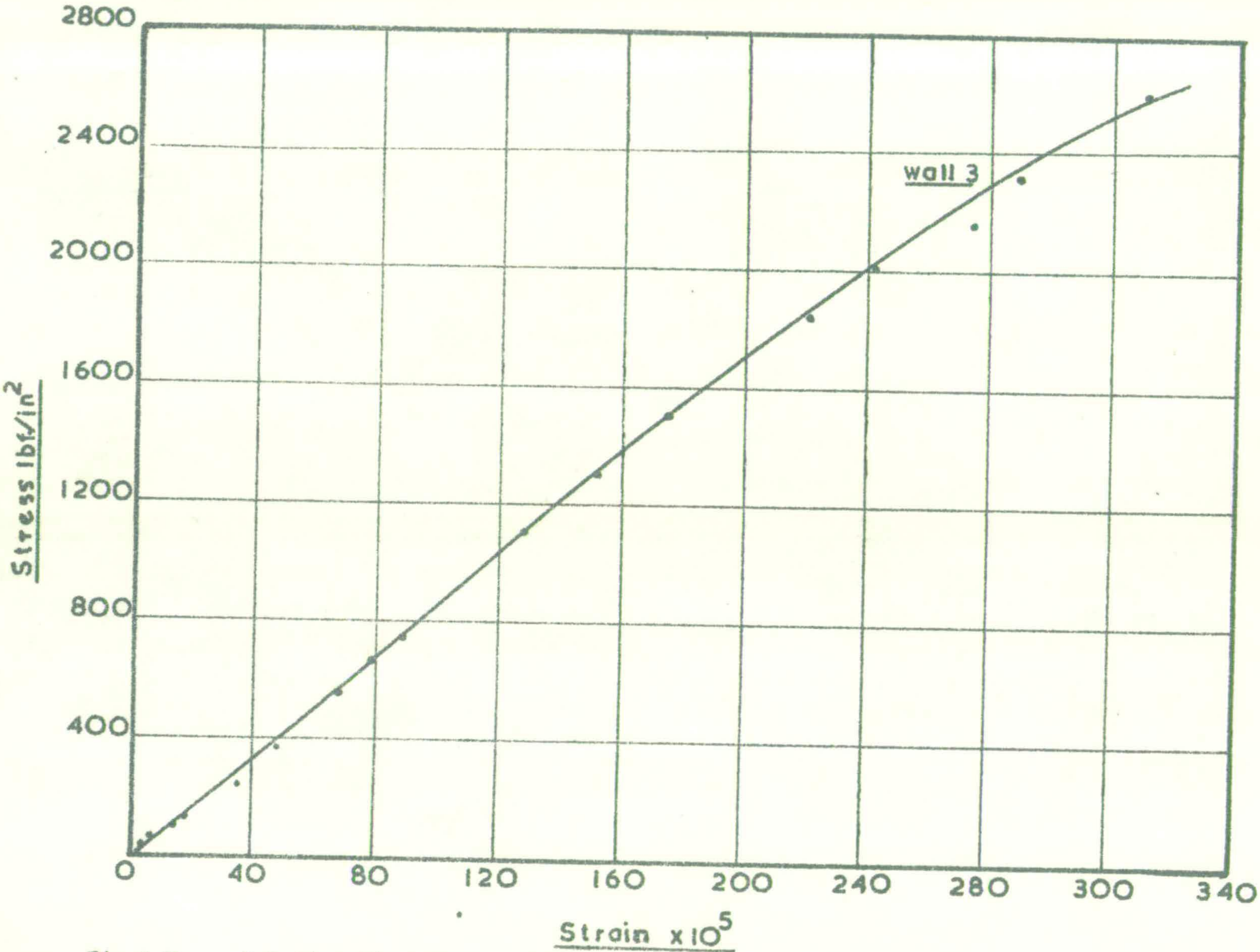


Fig.3.7. - Relationship between stress and strain (English Bond).

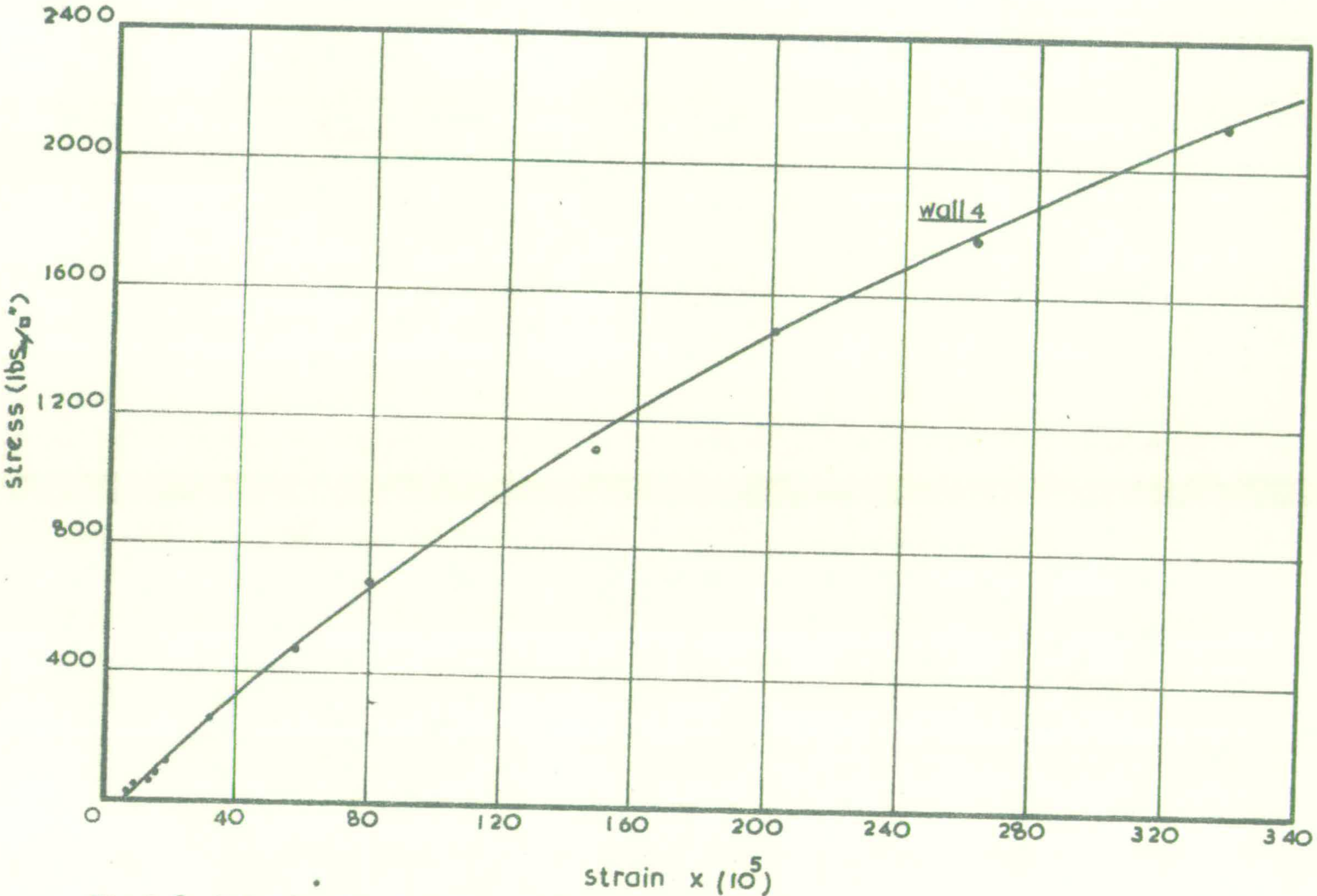


Fig.3.8 - Relationship between stress and strain (flemish bond).

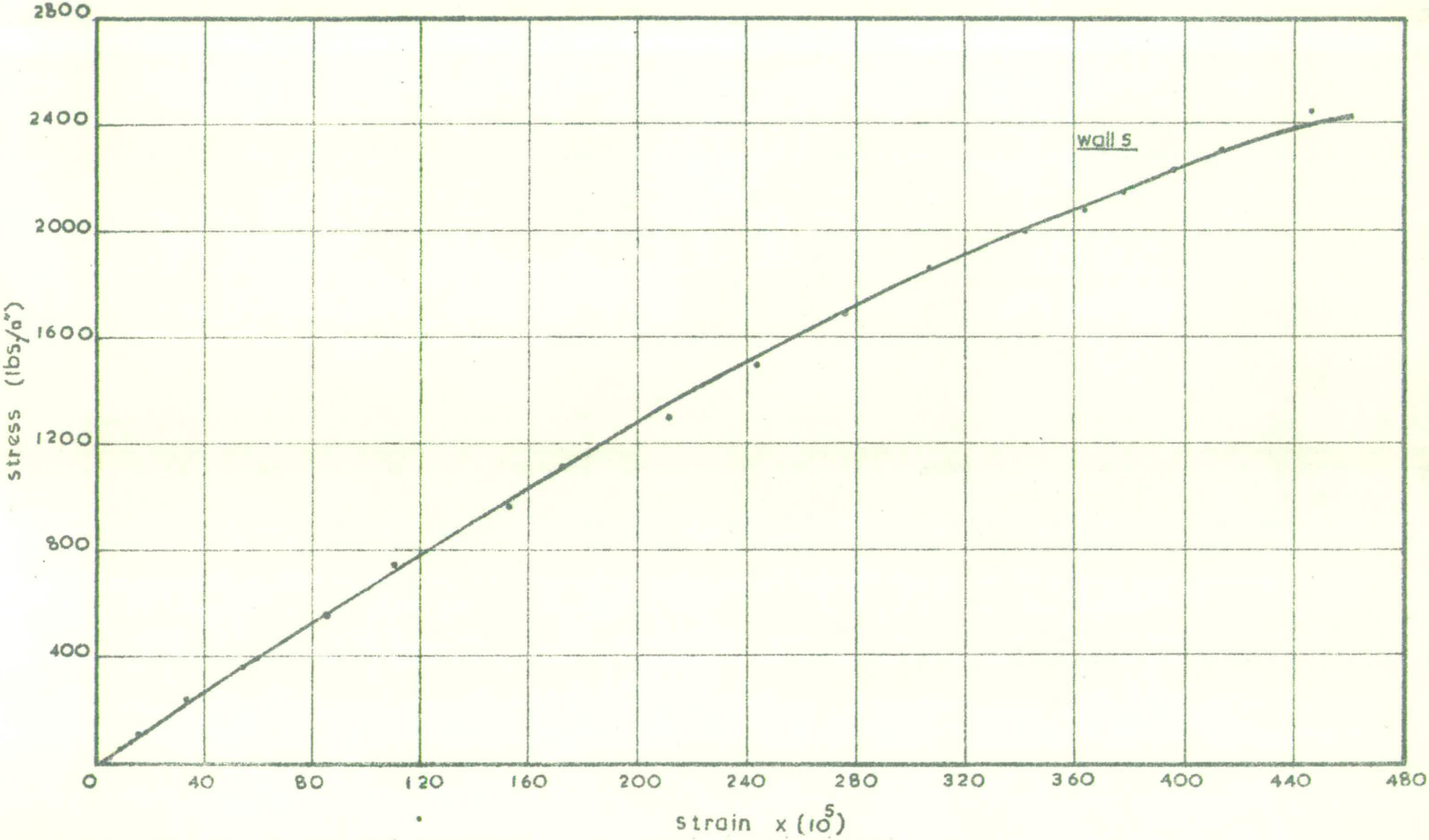


Fig.3.9 - Relationship between stress and strain (Flemish bond)

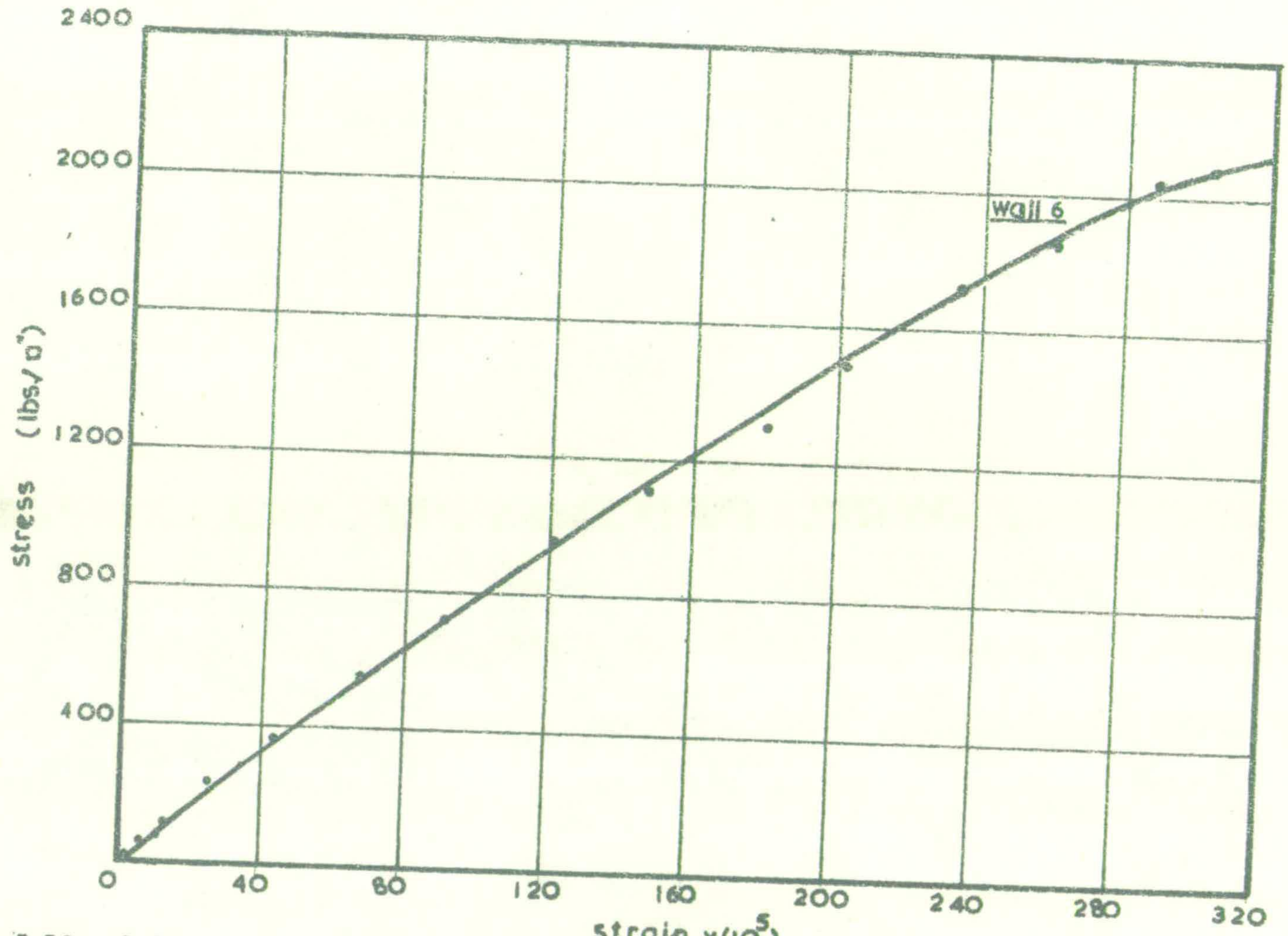


Fig. 3.10 Relationship between stress and strain (Garden Bond).

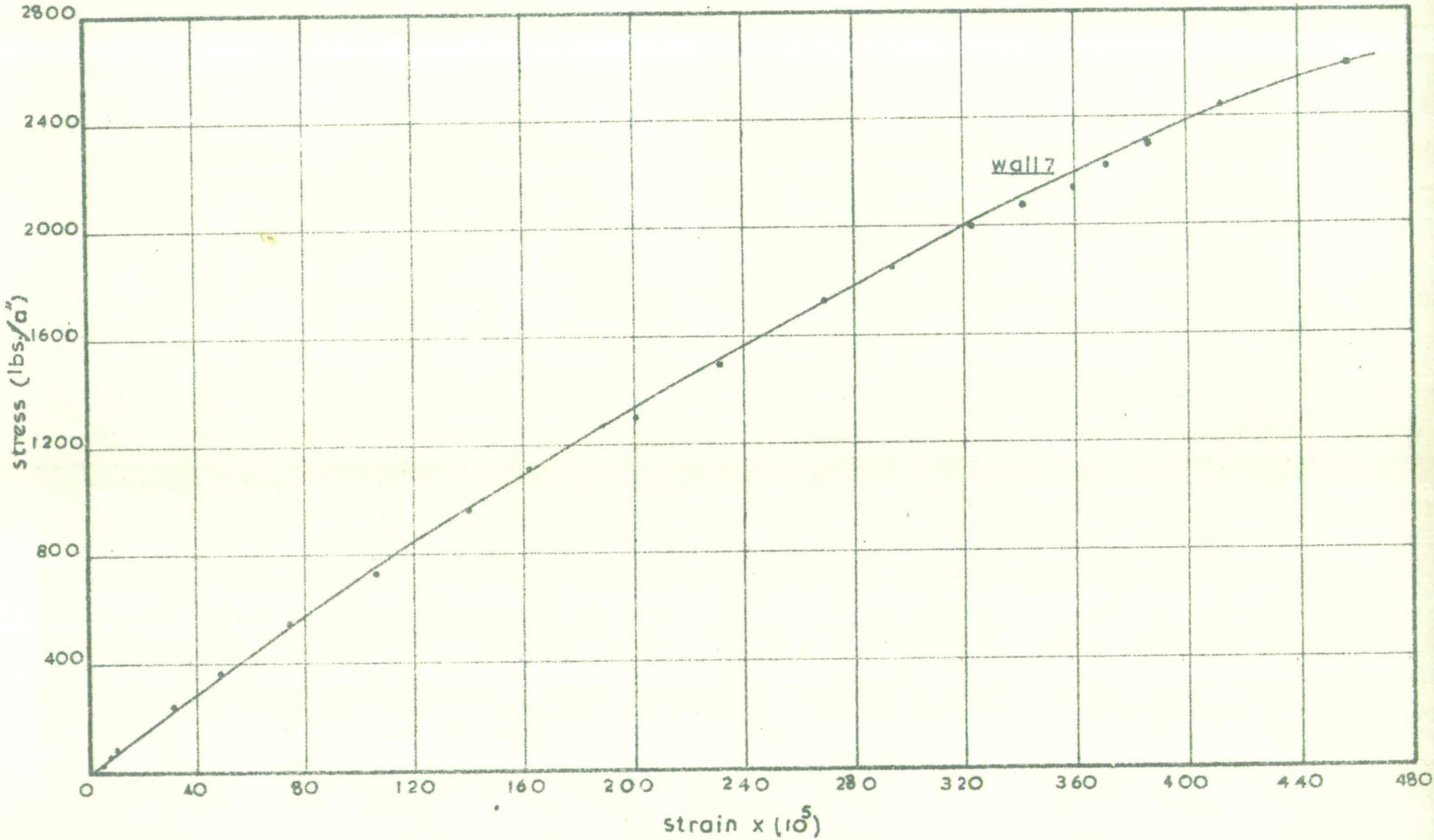


FIG. 3.11 Relationship between stress and strain (Garden Bond)

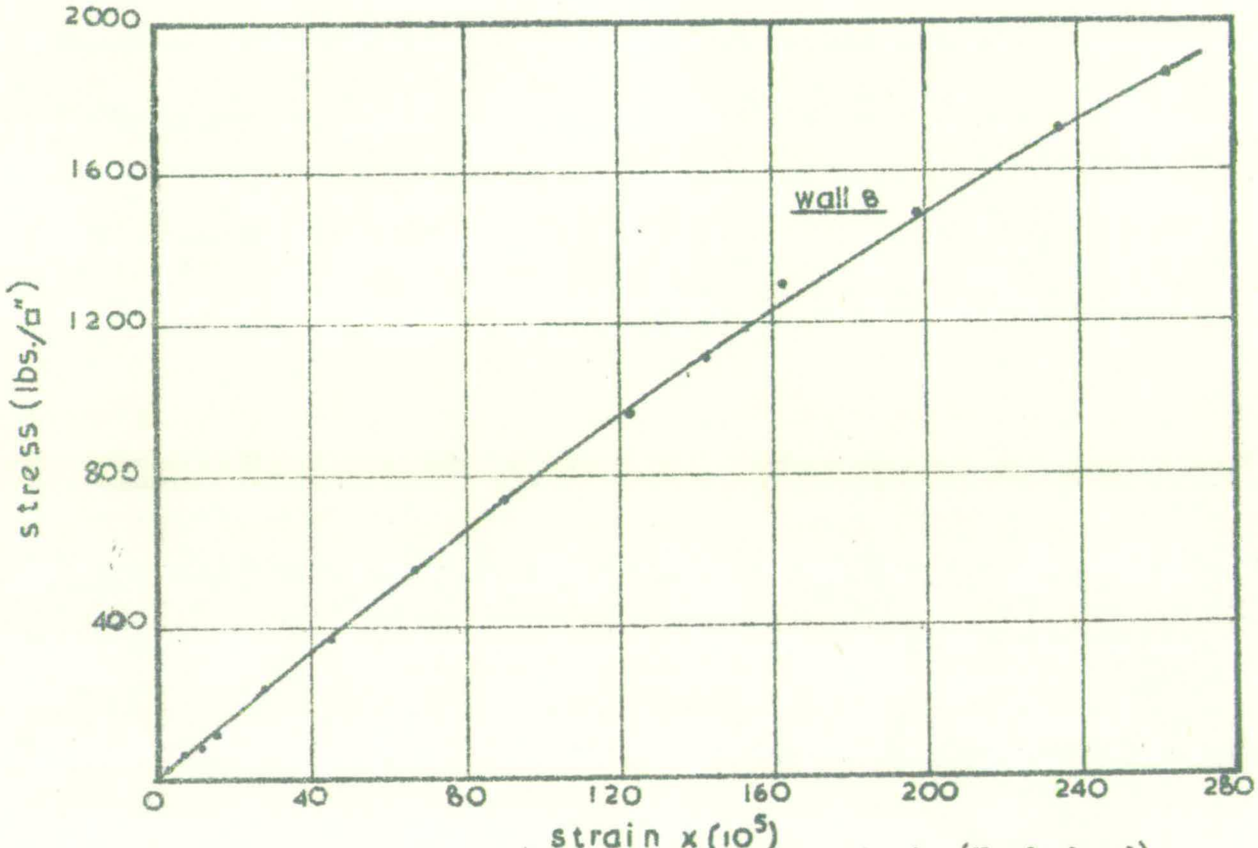


Fig. 3.12 Relationship between stress and strain (Headerbond)

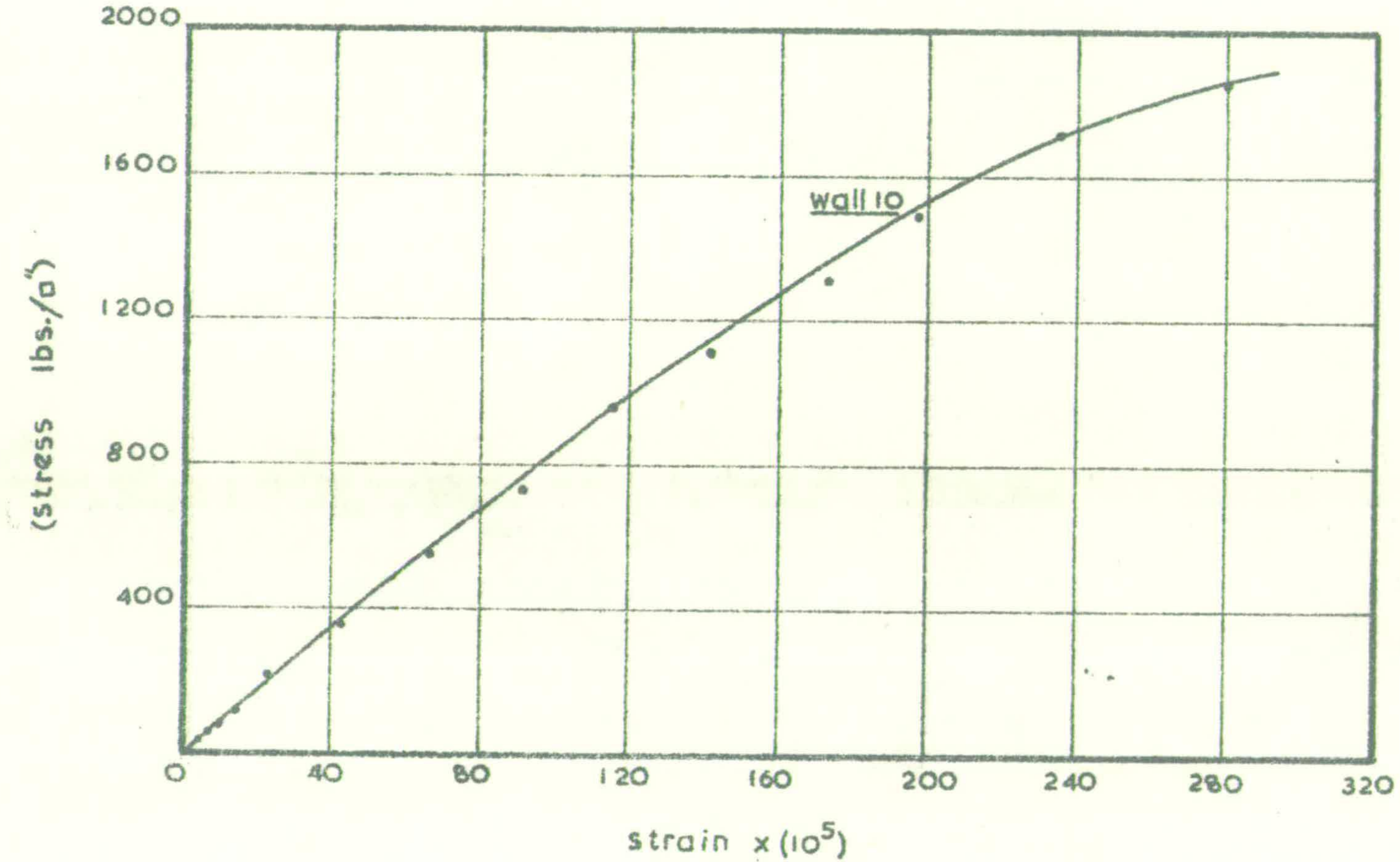


FIG. 3.13 Relationship between stress and strain (Stretcher bond without ties)

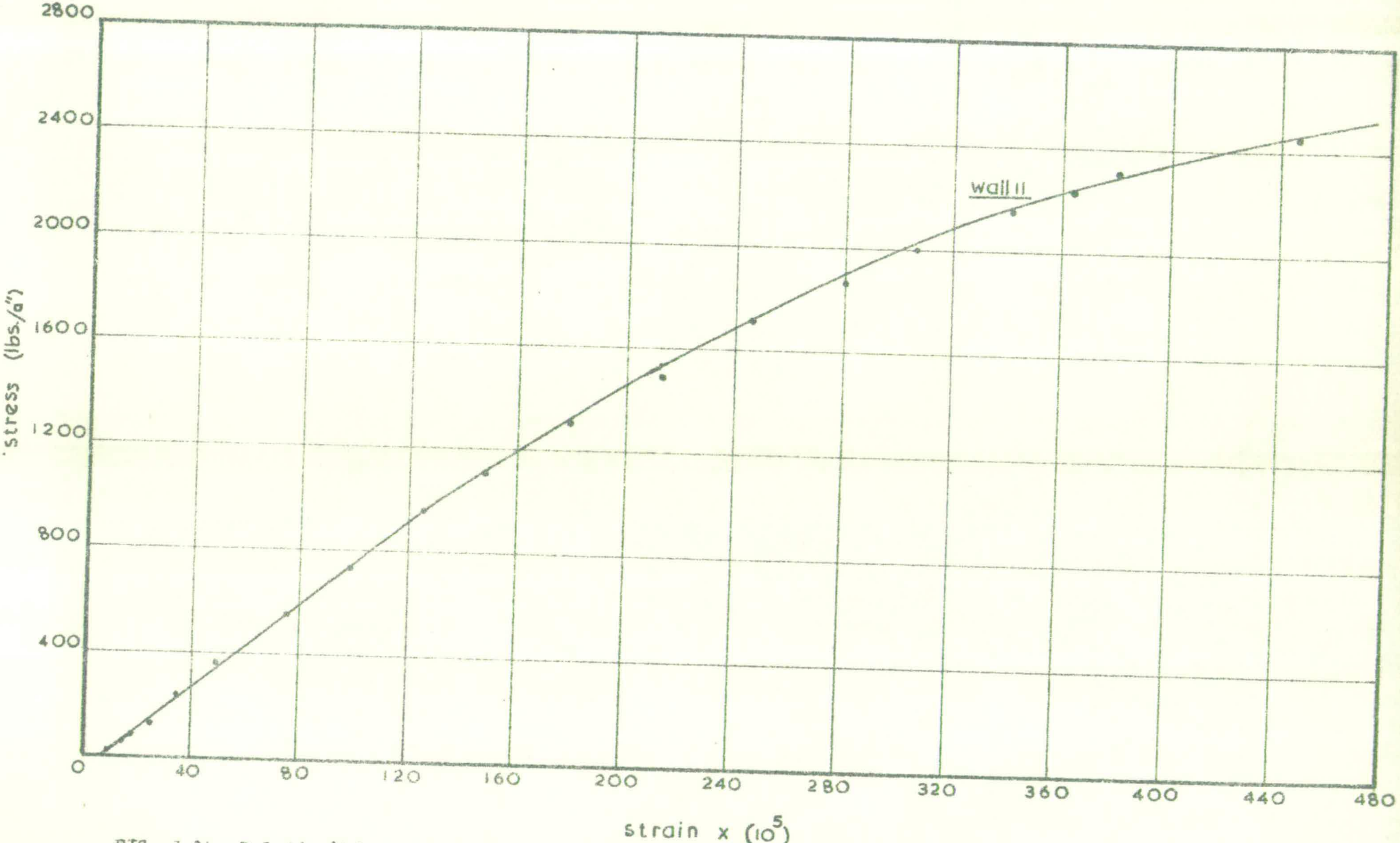


FIG. 3.14 Relationship between stress and strain (Stretcher bond without ties)

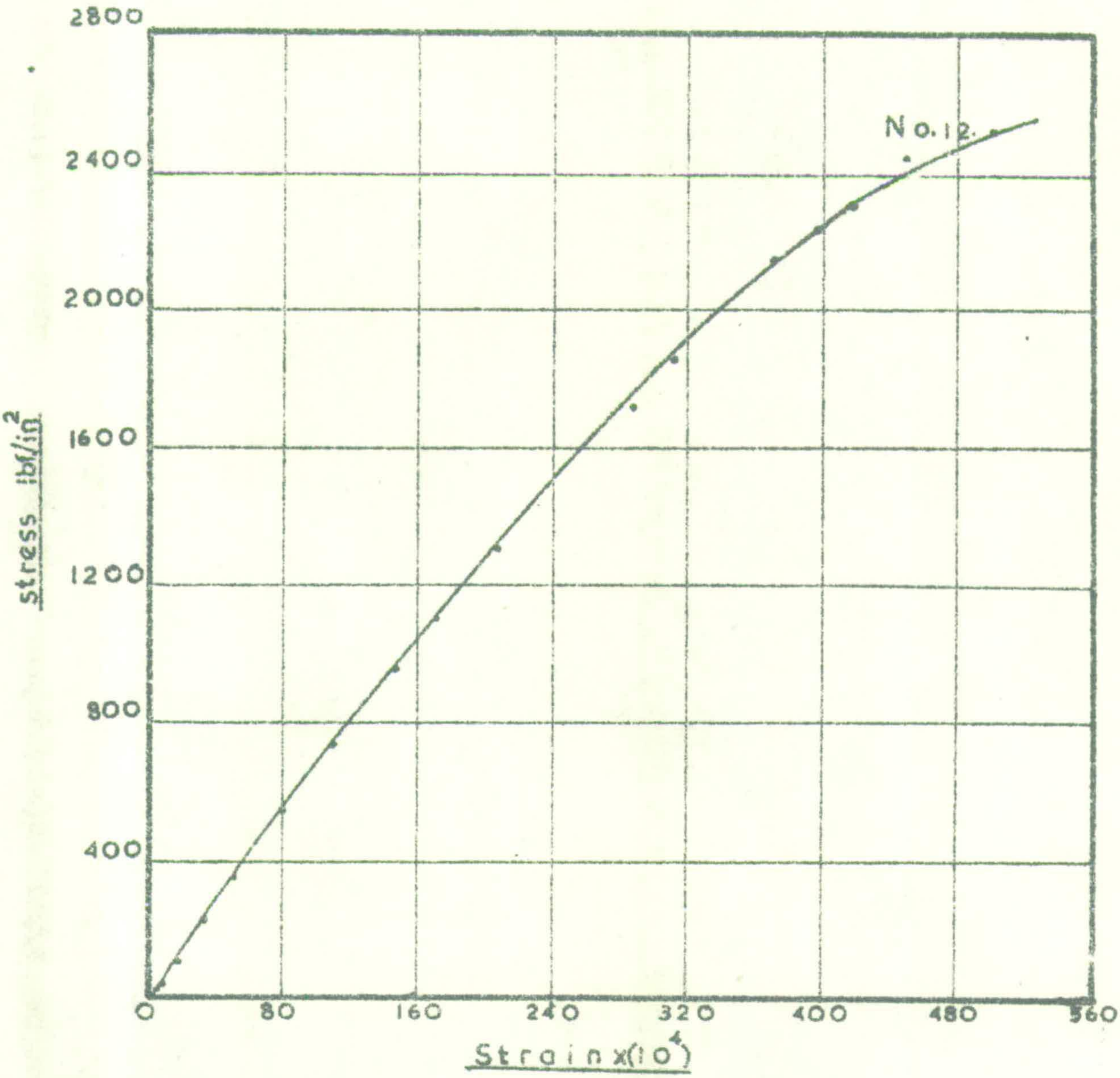


FIG. 3.15 Relationship between stress and strain (Stretcher bond with ties).

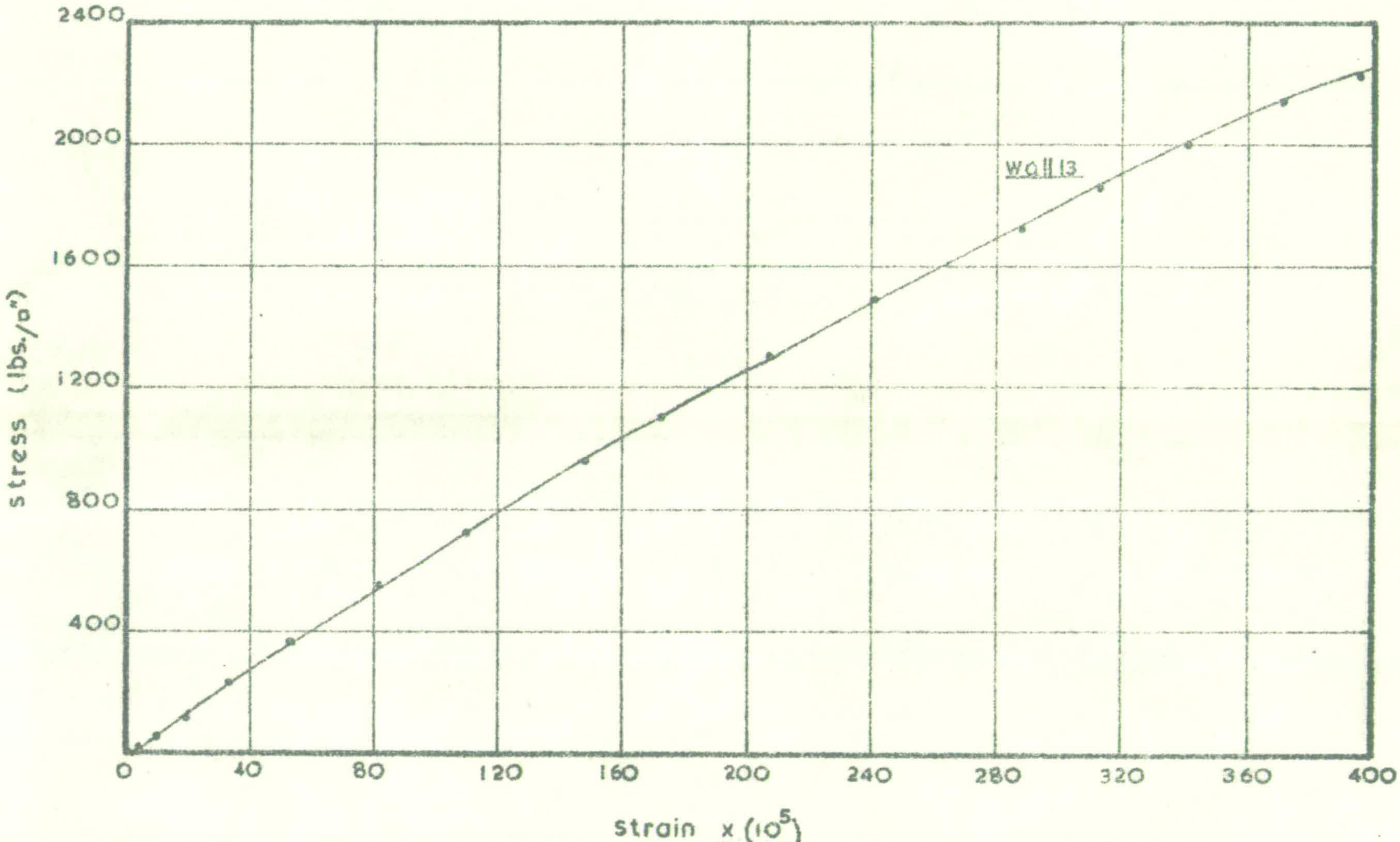


FIG. 3.16 Relationship between stress and strain (Stretcher bond with ties)

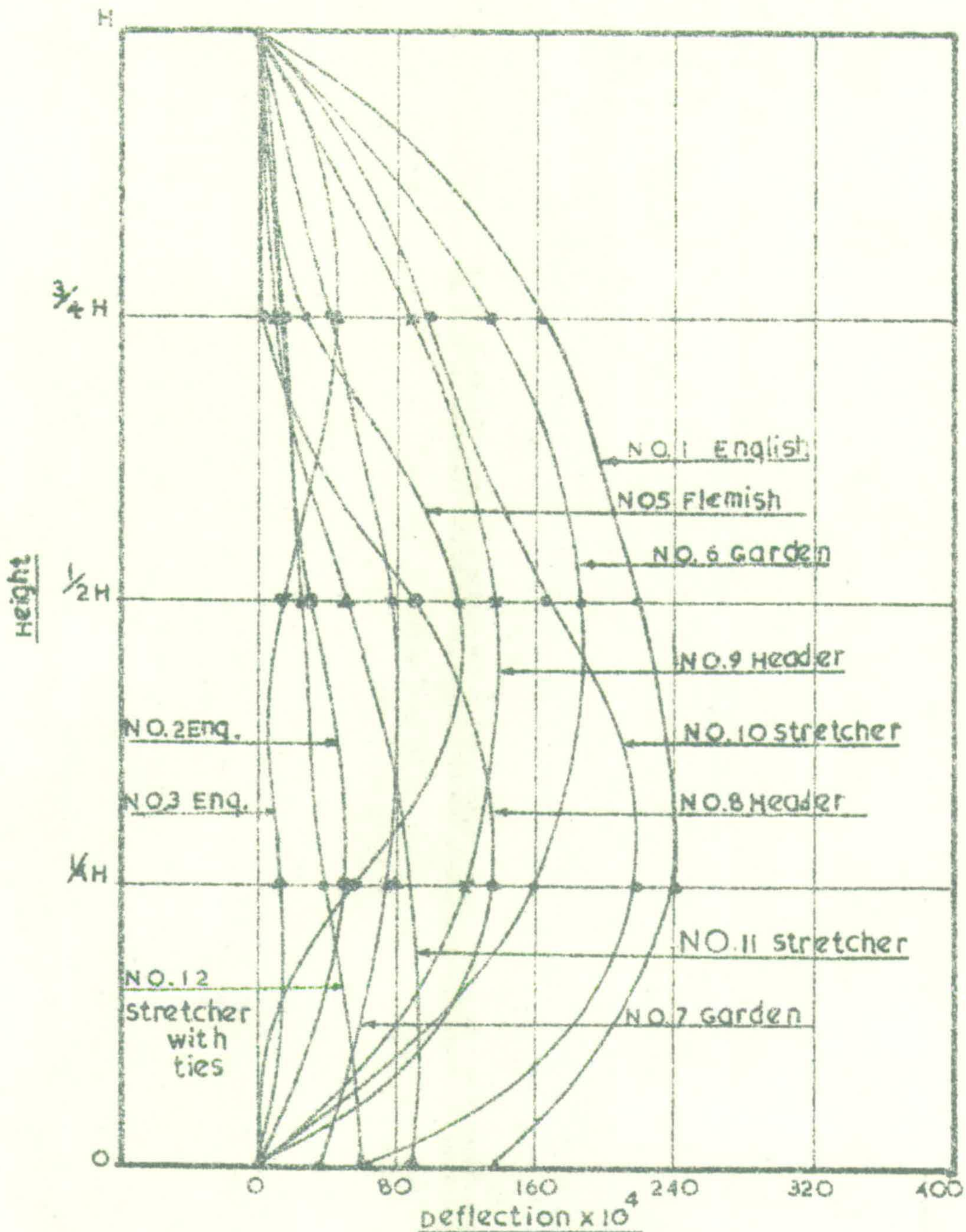


FIG. 3.17 Deflection Curves of the Walls in different bond at 737 lbf/in^2 stress.

investigated. ALBRECHT and SCHNEIDER,[1] investigated the relative strength of the English, header and garden bond in full-size walls, without providing slabs on top and bottom, and came to the conclusion that the load-carrying capacity of the brickwork is affected to some extent by the bond. Practically no difference was found in their test between the strength of English and garden bond, but the header bond was 8 percent stronger. However, this small variation may be accounted for simply as the scatter of experimental results. Recently in America tests[62] were done to investigate the relative strength of Garden and stretcher bond with ties, which indicate no significant difference between the ultimate compressive strength of the two, which confirms the finding of this work.

The Clay Products Technical Bureau[17] states that a 9-in. stretcher-bond wall will be weaker than an English or Flemish bond of the same thickness and could not be used for carrying superimposed loads from floors or upper storeys. During the present testing no significant difference in the load-carrying capacity of the stretcher bond was found when compared with others, but a 10 percent increase was developed by using wall ties.

There was no definite pattern of behaviour of walls as regards horizontal deflection. No two walls were found to deflect to the same extent. This may be due to the variation in the physical properties of mortar and bricks. There is also possibility of slight variation in workmanship. These factors will have profound effect on the flexural rigidity of the wall, which may vary from one course to another. The failure of the walls in general was due to transverse stresses, hence the ultimate load was not influenced by the magnitude of the lateral deflection.

From/

From the tests, it is also clear that once the mortar strength (1-in. cubes) reaches 800 lb/in^2 in model bricks, there is no significant difference in the brickwork strength. This agrees to some extent with the findings of DAVEY and THOMAS[19] in the full-size brickwork tests.

The ratio of brickwork-strength: brick-strength was found to vary from 0.48 to 0.67 and was in good agreement with American results[42,45] and somewhat higher than the results obtained for full size brickwork in this country. Some American tests[42] covered 302 full-size piers and found a variation of 0.31 to 0.72.

Although it has been widely reported that tensile splitting characterises the failure of brickwork in compression it is still the usual practice to relate the brickwork strength to the nominal strength of bricks in compression. It would appear more realistic and reasonable to relate the strength of brickwork to the tensile strength or modulus of rupture strength[57] of brick. The flexural tensile strength of the bricks is assumed to vary from 1/6th to 1/10th of the compressive strength and no definite[42] relationship has been established. The tensile strength does not increase proportionately with increase of compressive strength. To authors knowledge no test data are available on the tensile strength of bricks manufactured in this country. Probably, in some cases, bricks of compressive strengths 4000 lbf/in^2 and 6000 lbf/in^2 may have the same tensile strength as found in tests done in America[42] and the walls built from them under similar conditions might fail at the same ultimate stress, giving rise to higher value for the ratio of brickwork: brickstrength for 4000 lbf/in^2 brick than 6000lbf/in^2 brick. This may be the reason of getting higher/

higher ratio of brickwork: brickstrength in the model brickwalls than those of full-scale tests[8,19,54] done in this country, where it varies only from .29 to .52. Haller[28] has also concluded that the strength of brickwork increases with the increase in tensile strength.

From the tests, the safety factor in the code of practice[10] was calculated as 11; if applied to the model walls. The code is thus ultra-conservative.

3.6.1 COMPRESSIVE STRESS: MODULUS-OF-ELASTICITY RELATIONSHIPS

The stress and strain curves were not linear and hence the modulus of elasticity referred to is the secant modulus of elasticity. The value of the secant modulus decreases with the increase of load as represented by the formula and shown in Fig. 3.18.

$E = 763\,971\ (\pm\ 13\,895) - 70.05\sigma$ where E is the secant modulus of elasticity and σ is compressive stress (lb/in^2). This has been worked out by fitting linear equation to the test data of all the walls and it appears from Fig.3.18 that most of the experimental results are within two times the standard deviation (95% confindence limit).

At lower stresses, the value of the secant modulus of elasticity is somewhat irregular. This may be due to the closing of shrinkage cracks in the mortar and the seating of the material at the bond interface between brick and mortar during application of the load and may be ignored.

3.6.2. COMPARISON OF RESULTS - DOUBLE-AND SINGLE-LEAF MODEL AND FULL SIZE WALLS

The test results of the single-leaf model walls (one-sixth-scale representation of $4\frac{1}{2}$-in. full size) and full-scale tests on $4\frac{1}{2}$-in. brick walls/

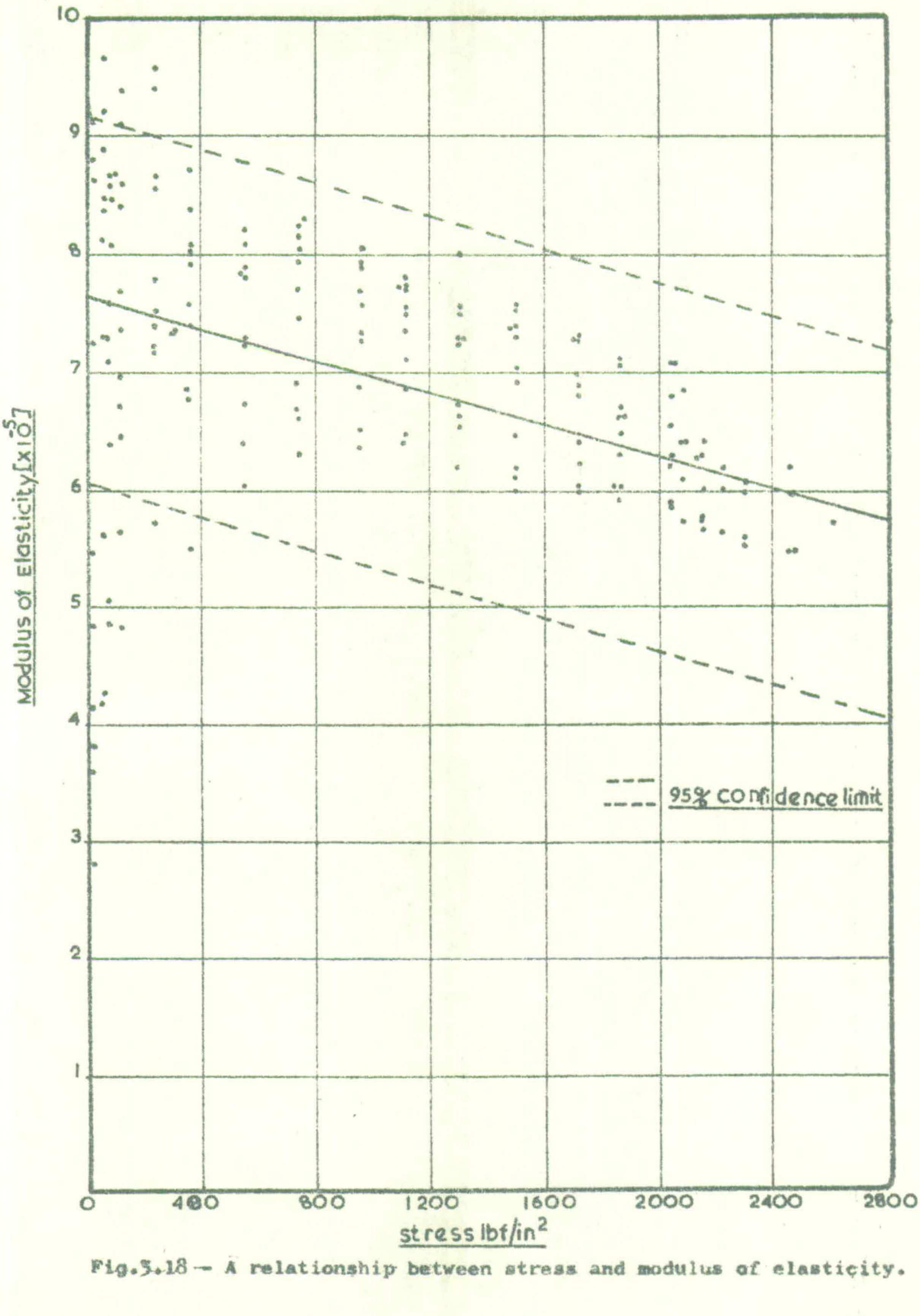


Fig.3.18 — A relationship between stress and modulus of elasticity.

walls carried out by PRASAN and others[47] may be compared with double-leaf model walls (one-sixth-scale representation of 9-in. full size) with the object of finding the relative strength of single-leaf walls and double-leaf walls of the same slenderness ratio.

The mortar strength in all these tests was quite high, and hence its effect on the brickwork strength is negligible. It has already been established that different brickwork bonds and the super loading[39,41,47] of slabs do not affect the ultimate strength of the brickwork, thus it is easier to compare the test results of MURTHY and HENDRY[39,41] and of PRASAN and others[47] with those obtained in the tests reported in this chapter.

Axial compression tests, without providing top and bottom slabs, have been conducted by SIMMS[54] on 9-in. full-scale walls built with 3800 and 4310 lb/in^2 bricks in 1:3 cement: sand mortar. The ratio of the strength of walls built with 3800 lb/in^2 to those built with 4300 lb/in^2 was found to be 0.92. This reduction factor was assumed to hold good between the model bricks of 3815 lb/in^2 and 4227 lb/in^2 used for single-leaf and double-leaf walls respectively.

Similarly, from the relationship established by DAVEY and THOMAS[19] between the brick strength and the strength of brickwork piers built with 1:3 cement: sand mortar (1.5 x 1.5 x 8-ft.high), the reduction factors for the piers built with 4227 lb/in^2 to those built with 5640 lb/in^2 and 7500 lb/in^2 are 0.769 and 0.625 respectively.

Using these factors and the reduction specified in the code for slenderness an attempt was made in Table 3.5 and 3.6 (P. 25 and 26) to compare the results/

results of one-sixth-scale representation of $4\frac{1}{2}$-in. and 9-in. walls and $4\frac{1}{2}$-in. walls in full scale.

It is clear from Tables 3.5 and 3.6, that there is good agreement between the predicted ultimate strength of single-leaf walls both in one-sixth-scale model and in full size, based on the tests done on the model double-leaf wall in one-sixth-scale. The highest ultimate strength of the single-leaf model wall was 1748 lb/in^2, which is well within the 95 percent confidence limit. It is also clear from Table 3.6 that the strength of 9-in. full-size walls built with 4227 lb/in^2 brick may be predicted from those built with the same strength of model bricks. From the results, it appears that in axial compression the load-carrying capacity of double-leaf bonded or unbonded walls is the same as that of a single-leaf wall of the same slenderness ratio.

3.7 CONCLUSIONS

1. The load-bearing capacity of the model brickwork is not significantly affected by the brick bonds investigated. The stretcher bond with ties carried 10 percent more load than without ties.
2. The typical mode of failure by vertical splitting (Plate 3.5 to 3.8) indicates that the tensile strength of the individual brick is important in determining the strength of the brickwork in compression.
3. There will be no appreciable increases on the load-bearing capacity of the model brickwork once the strength of 1-in. mortar cube reaches 800 lb/in^2.
4. The allowable stress according to the code[10] appears to be very conservative/

conservative and may require revision to take into account restraint from the concrete slabs.

5. The secant modulus of elasticity decreases with the increase of stress and statistically the relationship is linear (Figure 3.18).

6. The load-carrying capacity of double-leaf bonded or unbonded walls of two skins of $4\frac{1}{2}$-in. stretcher bond, is the same as that of a single-leaf wall of the same slenderness ratio (Tables 3.5 and 3.6); (Pages 25 and 26).

TABLE 3.1

DIMENSIONS OF MODEL BRICKS

Crushing strength (lb/in^2)	Dimensions (in.)	Range (in.)	Mean (in.)	Standard deviation	No. of bricks tested
4227	Length	1.457 - 1.470	1.465	0.016	12
	Width	0.681 - 0.693	0.687	0.0038	12
	Height	0.453 - 0.472	0.467	0.005	12

TABLE 3.2

PHYSICAL PROPERTIES OF BRICKS

Physical properties	Range (lb/in^2)	Mean (lb/in^2)	Standard deviation	Coeff. of variation (%)	No. of bricks tested
Compressive strength	3710.8 - 4488.6	4227	205	4.85	12
Tensile strength (axial)	358 - 235	294	-	-	7
Tensile strength (flexure)	662 - 850	773	-	-	7
Water absorption					
Wt. % after 24 h	12.70 - 12.05	12.31	0.18	1.46	12
Vol. % after 24 h	24.0 - 25.6	24.73	0.48	1.74	12
Wt. % after 5 h boiling	13.5 - 13.8	13.65	0.08	0.585	12
Vol. % after 5 h boiling	27.6 - 26.80	27.40	0.24	0.875	12
Saturation coeff.	0.926 - 0.88	0.899	0.014	1.56	12

TABLE 3.3

SIEVE ANALYSIS (B.S. 1200:1955) OF

LEIGHTON BUZZARD SAND NO. 19

B.S. Sieve No.	% passing by weight
14	100
25	95.7
52	9.45
100	1.74

TABLE 3.4

RESULTS OF THE INVESTIGATION OF BOND IN BRICKWORK

Test No.	Type of bond	Brick strength (lb/in²)	Mortar strength (1-in. cubes) (lb/in²)	Ultimate load (tons)	Ultimate stress (lb/in²)	Average stress (lb/in²)	Safety factor	Modulus of elasticity x 10^{-5} (800 lb/in²)		Brickwork-strength: Brick-strength
								secant	tangent	
1	English	4227	1642	10.8	2640	2510	11.7	6.27	5.47	0.624
2			1940	8.8	2150			6.52	5.92	0.508
3			1380	11.2	2740			8.20	8.20	0.648
4	Flemish		2350	8.8	2150	2375	11.0	6.55	5.42	0.508
5			1120	10.6	2600			5.42	5.10	0.615
6	Garden		1904	8.8	2150	2485	11.5	6.15	5.50	0.508
7			1315	11.5	2820			5.70	5.35	0.667
8	Header		1926	8.8	2150	2375	11.0	6.40	6.40	0.508
9			815	10.6	2600			-	-	0.615
10	Stretcher		2090	8.2	2040	2245	10.4	6.40	5.50	0.48
11			1092	10.2	2450			6.15	6.15	0.58
12	Stretcher with ties		918	10.3	2520	2485	11.5	5.30	5.05	0.596
13			784	10.2	2450			6.10	5.15	0.58

Note: Permissible stress in full-size brickwork according to C.P. 111: 1964 (as per clause 315) after reductions is 215 lb/in².

TABLE 3.5

COMPARISON BETWEEN THE RESULTS OF TESTS ON ONE-SIXTH-SCALE MODEL, SINGLE-LEAF AND DOUBLE-LEAF WALLS

Test No.*	Double-leaf walls (slenderness ratio = 8)			Single-leaf walls (slenderness ratio = 16)		Single-leaf walls (slenderness ratio = 16)		
	Brick strength (lb/in^2)	Brickwork strength (lb/in^2)	Expected brickwork strength for model bricks of 3815 (lb/in^2) bricks	Expected brickwork strength for model walls built with 3815 lb/in^2 bricks	95% confidence limit	Test results of Murthy's on single-leaf walls built with 3815 lbf/in^2 bricks	Average	MURTHY'S[39] Test No.
10		2040	1877	1183	$1434 \pm 2 \times 168$	945		4
11,13		2450	2254	1421		1417		3
12		2520	2318	1461		1420		5
2,4,6,8	4227	2150	1978	1247		-	1482	-
9,5		2660	2392	1508		1748		6
3		2740	2419	1589		1665		7
7		2820	2549	1635		1700		2

*As in Table 3.4

TABLE 3.6

COMPARISON BETWEEN THE RESULTS OF TESTS ON ONE-SIXTH-SCALE 9-in. BONDED MODEL WALLS AND ESTIMATED STRENGTH BASED ON TESTS ON $4\frac{1}{2}$-in. FULL-SIZE WALLS.

		$4\frac{1}{2}$-in. full-size wall (slenderness ratio = 16)		Double-leaf wall (slenderness ratio = 8)		
Test No.*	Brick strength (lb/in^2)	(Test results) Brickwork strength (lb/in^2)	Expected strength of similar wall built with 4227 lb/in^2 bricks	Expected strength of 9 in. wall built with 4227 lb/in^2 bricks	Strength of one-sixth-scale model wall representing 9 in. full size built with 4227 lb/in^2 bricks	PRASAD Test No.
13	5640	2040	1610	2543	2450	2
3	5640	2205	1740	2749	2740	3
5,9	5640	2080	1642	2594	2600	5
11	5640	1945	1535	2425	2450	6
12	7500	2580	1612	2546	2520	7
2,4,6,8	7500	2150	1344	2123	2150	8

*As in Table 3.4

CHAPTER 4

A survey of similar work on shear wall structures.

4.1. INTRODUCTION:- Until recent years brickwork was never analysed and designed on scientific principles, but was regarded as a traditional piece of craftmanship and designed by purely emperical procedures. Since the construction in Switzerland of 18-storey[27] high building in brickwork with relatively thin walls, considerable interest has been aroused in different parts of the world in the revival of multi-storey load bearing brickwork structures stiffened by shearwalls. However, very little work has been done on multi-storey brick cross-wall structures in general and practically nothing is available as regards to the behaviour of single or multi-storey, interconnected brick shear walls. On the other hand, a similar problem has been encountered in concrete construction and this has resulted in a good deal of analytical work which will be very briefly surveyed in this chapter.

4.2. METHOD OF ANALYSIS FOR MULTI-STOREY SHEAR WALLS CONTAINING OPENINGS:- Generally, two methods have been used for the analysis of shear wall containing openings:

1. Continuum approach 2. Frame analogy.

4.2.1. CONTINUUM APPROACH:- The beams connecting the walls are replaced by an equivalent continuous medium. Further assumptions are made that the beam has a point of contraflexure in the centre and axial and shear deformation are negligible. On this basis a second order differential equation/

equation is obtained to determine the redundent forces in the system.

Chitty[15](1947) appears to be the first to use this technique for analysing the problem of tall buildings composed of any numbers of column and rigidly connected beams under wind loading. The approximate general solution for (n + 1) number of columns of constant and variable cross-sections have been developed neglecting the axial deformation. However, the axial deformation was accounted for in the case of two equal columns of constant cross-section.

Later on, the same technique was extended to take into account the axial deformation of wall and applied to shear walls containing openings by Shulz[52](1961), Errikson[24](1961), Beck[4](1962), Rosman[49] (1962), Magnus[35](1965). Basically, the approach of all these authors are the same except for the choice of the redundant function. In case of all but Schulz and Magnus the redundant function was shear in the continuum. Schulz has taken axial force, whereas Magnus's the variable is strain in the wall. The paper by Magnus is of much practical importance as design charts have been prepared, which enables one to find out internal forces, bending moments and the horizontal deflection of walls. Based on Rosman's[49] approach, very recently Coull and Chowdhury[18](1967) have also presented design charts for rapid evaluation of the maximum deflection and stress in the inter-connected shear wall.

The governing differential equation of shear walls containing openings becomes very cumbersome and tedious for more than three wall sections. Soane[59](1966) has suggested analogue simulation for the solution of the general/

general equation. On the experimental side his work appears to be the first and most comprehensive and was applied to the design of a 14-storey load-bearing brickwork structure (Plate 4.1) under wind loading. For this particular building the actual three dimensional wall complex has been replaced by a row of hypothetical walls with appropriate centroid position, areas and moments of inertia. The theoretical solution was compared with the test results obtained from a 1:48 scale model made of perspex. The stress distribution obtained analytically was in good agreement with those obtained from strain gauges fixed on the models. The test demonstrated co-action between the walls and floor slabs. The results are quite interesting, but it would be difficult to apply this to brick structures as a routine procedure. Further, Soane's[59] model was made of honogenous elastic material and joined with glue which makes a perfect rigid joint capable of transferring the bending moments, whereas brickwork is neither homogeneous nor perfectly elastic and joints are more flexible. However, the above work gives an insight to the actual behaviour of an idealised multistorey shear wall structure. Barnard and Schwaighofer[3] (1966), have done some model studies made out of epoxy sheets to establish the width of the slab interacting with coupled shear wall and suggested some approximation for the solution of Rosman's theory, which is very convenient for the design office work.

4.2.2. <u>FRAME ANALOGY:-</u> In this approach shear wall is idealised and replaced by an equivalent frame. The centre line of the wall becomes/

Plate 4.1 - Essex University Tower (14 storey high) - England.

becomes the centre line of the column and height becomes the distance between the centre line of the openings.

Green[26](1952) and Amartunga[2](1962) suggested this method for analysis of shear wall with opening. Green has assumed the point of centraflexure in the centre of columns and beams like that of Portal method for analysing concrete bracing walls, which is not very realistic in approach. Though Clark's[16] method does not fall in this category, his laminated beam approach seems more reasonable. His assumption is that the laminae are connected with number of rivets or fixed at number of points. The rivets resist the longitudinal shear and redistribute it to the laminae. The area moment method of analysis is used, which gives the slope and deflection between two points. However, it requires extensive algebraic computation and could not be extended to multi-storey multibay shear wall structures. Amartunga[2] used the flexibility method for the analysis of the equivalent frame. Araldite model was used for photoelastic test to find out the stress distribution around the opening and perspex model was used for deflection for comparison with the theoretical analysis. By a similar analytical method Candy[12](1964) has analysed coupled shear wall by computer using the slope deflection method, which takes into account bending, shear and axial deformation of the wall.

Frischmann, Prabhu and Toppler[25](1963) suggested two different methods for analysing the interconnected shear walls based on the analysis of rigidly jointed frameworks. In the first place the structure is replaced by an equivalent single column of moment of inertia equal to the sum/

sum of all the columns of inertia and the beams are replaced by the sum of restraint moments applied to each floor. For simplication, the system of forces are distributed throughout the whole length of structure and second order differential equation in terms of column bending moments are obtained. However, the axial deformations of beam and column and shear deformations have been ignored. The influence coefficient method was used in the second case with the only modification that between the centroid and the face of the shear walls the connecting members have infinite stiffness.

Macleod[34] (1966) has used all the existing methods (continuum approach, wide Column beam, grid and finite element) for the estimation of the stiffness of simple shear wall pierced to a regular pattern with regard to the size and position of the openings. He developed the computer programme on KDF9 digital computer. The results of analysis were compared to those obtained from the test conducted on aluminium model and generally good agreement was found.

All the above methods of analysis of multi-storey shear walls containing openings assume linear elastic behaviour. The experimental studies were also done on perfectly elastic and homogenous material and it is doubtful whether this will hold good for multi-storey crosswall type of structure in brick.

To author's knowledge, the only available literature on the behaviour of multi-storey brick structure is of Murthy[39] (1964) and Murthy and Hendry (1966)[40] The aim was only to find out stiffening effect of shear walls in the multi-storey/

crosswall structure and the ultimate strength of the structure under wind loading. The shear walls were infilled at different stages of the test and it was found that the rigidity increased to 104 times the initial rigidity without the infill. The ultimate strength found was very high compared with a single storey test and this point remained unexplained.

4.3. STOREY HEIGHT SHEAR PANEL:-

The earliest investigation was conducted by Benjamin and Williams[5,6] (1951-56) in Stanford University. Most of the tests were conducted on brick shear walls with and without bounding frame and concrete shear walls with single and multiple opening. Based on these results in 1958, they suggested a simple formula based on strength of material for taking into account axial, bending and shear deformation in walls containing openings.. Further they have simplified the analysis in their recent book[7] (1959) and suggested that in case of brick shear walls only shear deflection should be considered and for regular multi-storey structures the single storey theory of simple cantilever holds good considering the highly variable nature of brick and mortar composite.

This approach is quite simple, but completely ignores composite action, which is not realistic. Only two brick shear walls without bounding frames were tested and very low ultimate stress was found. It is not surprising to the author as no precompression[58] was applied from the top. No further test was done and it appears that these investigators, were interested only in brickwork infill as a means of imparting lateral rigidity - rather than thinking in terms of crosswall structure.

Simms/

Simms[53] (1964) reported some racking tests on storey height walls made of solid and perforated bricks and hollow clay blocks. The test arrangements are as shown in Plate 4.2. The test arrangement No. 1 does not simulate the actual condition in a building of non framed type. However, there is some resemblence with the forces generated in infill panels. Though the second arrangement represents the effect of dead load on a multi-storey wall panel, the horizontal displacement could not take place without undue friction. However, the work was conducted very early and it served the purpose of providing some basis for the code in the absence of any data available at that time. The failure pattern was the same as one would expect in the case of a shear wall.

Some photoelastic tests were done by Amartunga[2] (1962) in single panel containing an opening to find out the stress distribution near it. Kazimi[32] has also used photoelastic models to study the stress distribution in single storey shear walls with and without an opening supported on rigid and flexible base. The mathematical analysis has been presented using line solution technique. His work is of a highly theoretical nature and appears to have little practical application.

As explained earlier in this chapter, the theoretical analysis does not reflect the true behaviour of brick shear walls. In author's knowledge, the only work simulating the actual condition of shear wall in a building without openings is that of Murthy[39] (1964). The aim was to/

to study the ultimate behaviour under wind load with regard to precompression. Approximate formula was suggested based on couplet tests. At higher value of precompression the following formula has been suggested

$$\underset{\text{(bond shear)}}{V_b} = \underset{\text{(shear strength)}}{V_{ult}} - \overset{\text{(friction)}}{f\sigma} \leqslant 0 \quad \text{(fig. ..... 4.1)}$$

To the author's mind the bond shear is something analogous to the cohesion in soil, which is independant of vertical precompression and because of small numbers of tests carried out - Murthy[39] may have reached an incorrect conclusion. Working on the above assumption he has established a relationship as shown in fig.4.2 (Author's correction in dotted line). From it may be seen that the experimental result obtained in the case of storey height shear walls was not entirely due to experimental error, but reflects the actual behaviour of shear walls as explained in 6.7 (Fig.6.6.)

Some tests in pure shear have been conducted by Monk[38](1963) using 16" x 16" x 4" thick wallets in conventional mortar and with mortar added with 20% Saran Polymer. The racking resistance was found three to four times more in the case of mortar added with admixture to that of conventional mortar. Some test was done according to A.S.T.M. method of testing the wall in racking as shown in Plate 4.3. The racking resistance of 8' x 8' x 4" wall built with mortar mixed with Saran polymer was 302 psi, whereas the diagonal tensile strength obtained/

<u>4 - Plate 4.2 - 4.3</u>

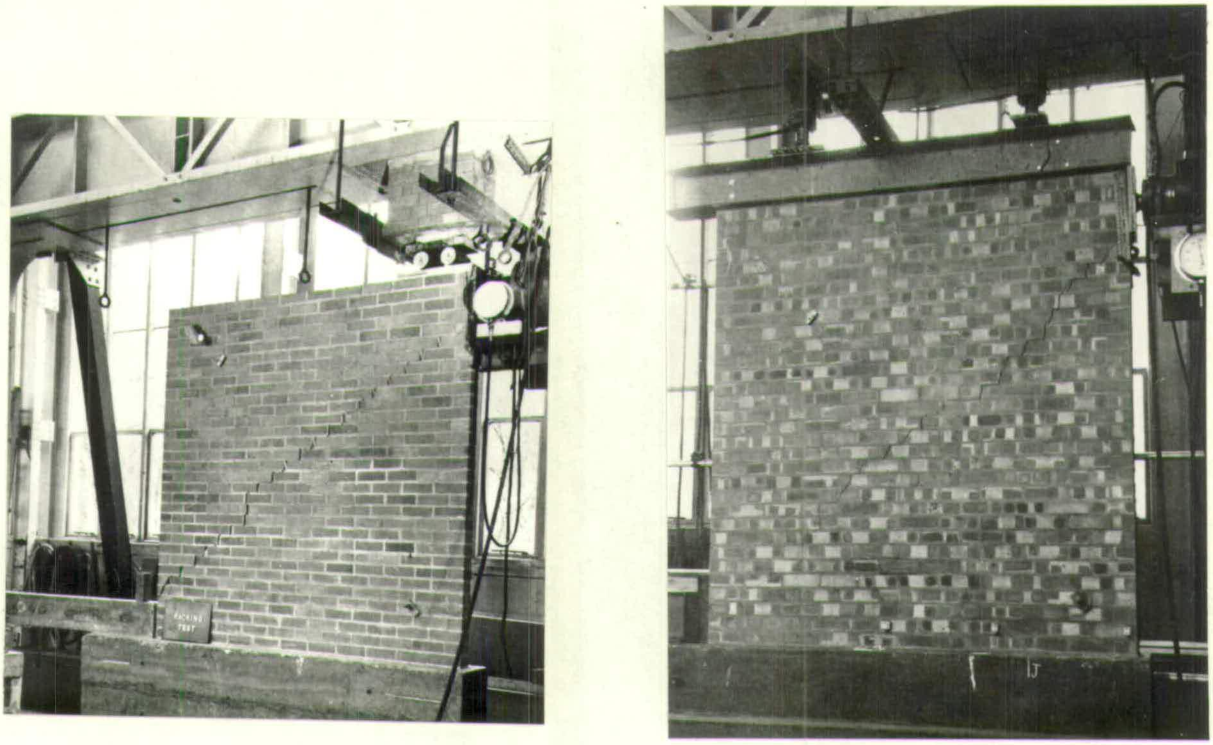


Plate 4.2 - Showing test arrangement of Simms.

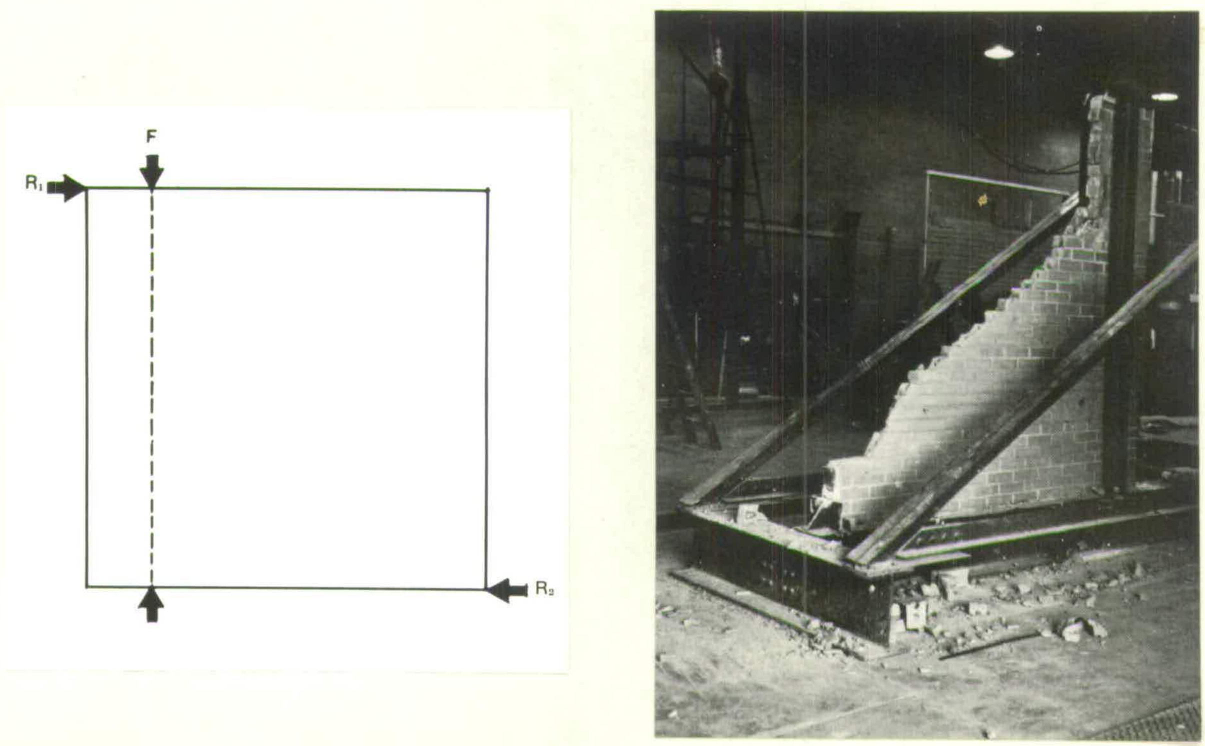


Plate 4.3 - Test arrangement of Monk.

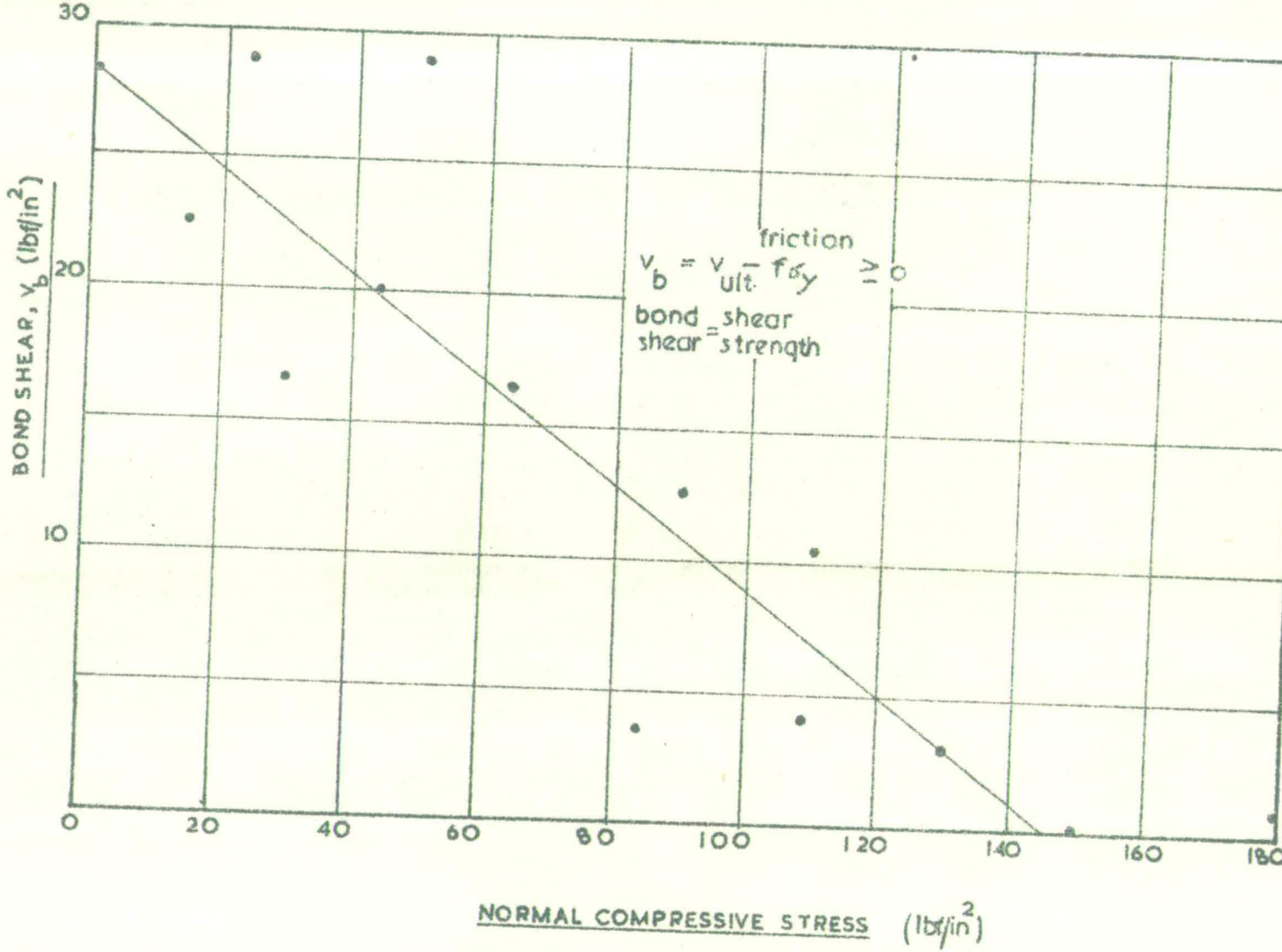


FIG. 4.1 RELATIONSHIP BETWEEN THE COMPRESSIVE STRESS AND BONDSHEAR.

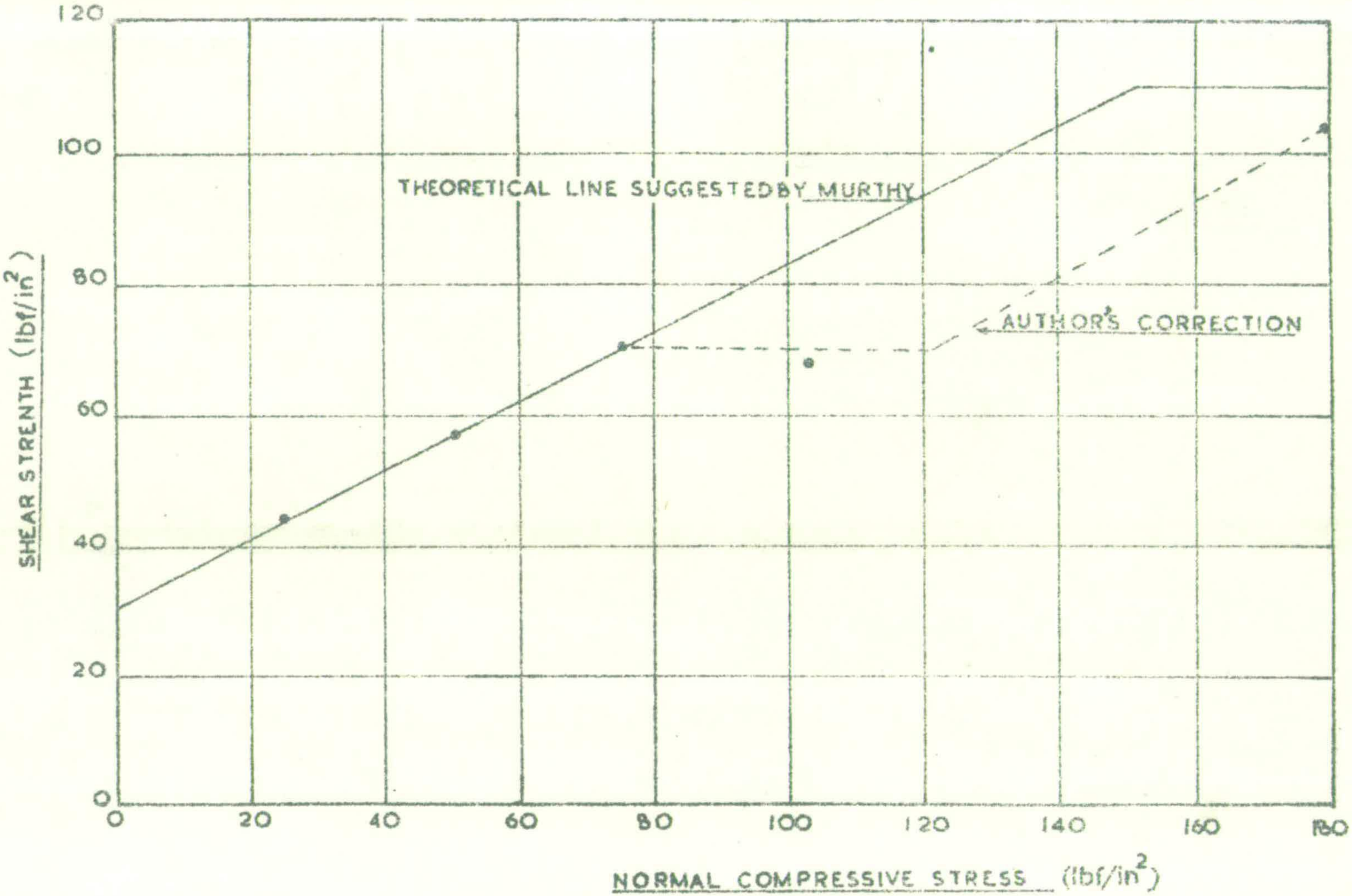


FIG. 4.2 SHEAR STRENTH OF STOREY-HEIGHT SHEARWALL STRUCTURE SUBJECTED TO PRE-COMPRESSION

obtained from 16" x 16" x 4" prism was 211 psi. The indeterminate tie force would certainly have had some influence on the ultimate strength, and the test conditions does not reflect the actual condition of a shear wall in a building. The value of diagonal tensile strength as obtained from wallet test may be of much importance for perforated bricks with S ırau polymer as used in this case, but the results cannot be applied universally[58] to all types of bricks as explained in Section 6.6, 6.7 and Fig.6.6.

Similar methods have been adopted by S.C.P.R.F.[42,62] (1965) for all their small specimen tests and as the tie rod imposed an indeterminate compressive force at the end of the wall it is doubtful whether the results could be applied and adopted for the ultimate strength of a panel in shear without precompression. Another promising method adopted by the S.C.P.R.F. is to test a small circular specimen as shown in fig. 6.7, which is more realistic assessment of diagonal tension or shear strength of the brickwork for the type of brick used by them.

Though racking tests have been done on full size brick panels and model single and multi-storey brickwalls, nothing is available for the design of complicated wall units under wind loading. A lot more research has to be done at full scale before any reasonable solution could be presented, which will account for the composite behaviour of the brick structure. Before embarking on the programme of testing full-size structure it is better to have some useful information from model testing.

CHAPTER 5

Investigations of Bond Tension, Bond Shear and the effect of pre-compression on the Shear strength of model brick masonry couplets.

5.1 INTRODUCTION:

One of the most important considerations for the design of brick masonry subjected to racking load is the strength of the bond between brick and mortar. In a brick wall subjected to racking load, failure of the brickwork can occur at the interface between brick and mortar, within the mortar joint or even within the bricks, whichever is the weaker. Failure at the brick mortar interface is the most common, although it is possible to increase the frictional resistance by precompression so that tensile failure occurs in the brick or in the mortar.

The investigation described in this chapter was carried out in two parts, the first involving a study of bond tension and bond shear including the influence of the moisture content of the brick on bond. The effect of vertical compression applied to the couplets during the curing period was also considered.

The second part (Section 5.6) considers the shear strength of brick couplets subjected to precompression normal to the bed joint.

5.2. MATERIALS

5.2.1. BRICKS

In all the tests described in this note, one-sixth-scale model bricks of average crushing strength 4332 lb/in^2 were used. The physical/

physical properties are given in Table 5.1.(P.45).

5.2.2. CEMENT

The compression tests on cement mortar cubes were conducted according to B.S. 12: 1958 and the results given in Table 5.2 show that the cement conforms to this standard.

5.2.3. SAND

The sand used was Leighton Buzzard No.19, the grading of which is shown in section 3.2.2 (Table 3.3).

5.2.4. MORTAR

'Ferrocrete' was used in making a 1:3 cement mortar; the average crushing strengths of 1-in. cubes of mortar at the time of testing the specimen were 1900, 2120, 1926 lb/in^2.

The water cement ratio (0.91) was adopted for all tests.

5.2.5. CURING

The same procedure of curing was adopted as mentioned in chapter 3 (section 3.2.5).

5.3. BOND TENSION TESTS

In adopting cross brick couplets for bond tension tests, POLYAKOV[46] found difficulty in fixing the application of load in the centre of the couplet, and also in laying the bricks. Hence he made the assembly in the form of a cube, shown in Fig. 5.1, made of two halves mortared together and pulled apart by special clamps.

However, other research workers, e.g. PEARSON[44] and KAMPF[31] found the test quite satisfactory and it is generally accepted that cross brick couplets give satisfactory results for bond in tension. Pearson used selected/

selected bricks for the top and for economy a common unselected brick for the bottom. Before placing the mortar for assembly he treated the lower brick with high early strength cement grout to ensure failure on top of the joint. This is, of course, contrary to practice at site. It would appear that Pearson's[44] results for bond tension were higher than normal because the mortar was prevented from losing any moisture to the bottom brick. DAVISON[20] found that the bond strength between the mortar joint and the upper brick was less than that between the mortar joint and the lower brick. He attributed this to the fact that before laying the upper brick moisture from the mortar will be absorbed into the lower brick. Therefore, the consistency of the mortar is less when the upper brick is laid due to loss of moisture.

In the bond tension tests described in this note two whole bricks were used and tensile load was applied as shown in fig. 5.2 and plate 5.1.

For a given type of brick and mortar the bond strength is affected by the moisture content of bricks at the time of laying. To investigate this, bricks were dipped in water for periods of time ranging from 5sec.to 2hrs. before the couplets for the tension and shear tests were made. The moisture content of each sample was determined and the results are shown in Table 5.3.P.47. From the tests (Fig. 5.4) it is clear that the moisture content of the bricks at the time of laying influences the bond strength of brickwork, though THOMAS and SIMMS[64] concluded from a small number of full-size tests that it was not a very important factor. The results agree/

5 - Plate 5.1

Plate 5.1 - Test arrangement for finding tensile bond strength.

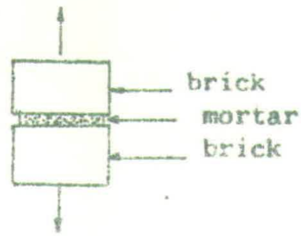


Fig.5.1 Bond tension test arrangement by Polyakov

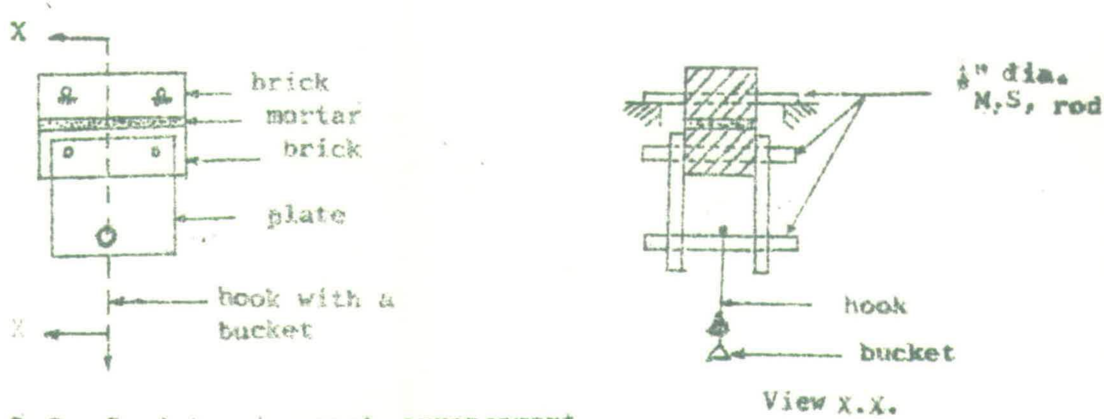


Fig. 5.2 Bond tension test arrangement

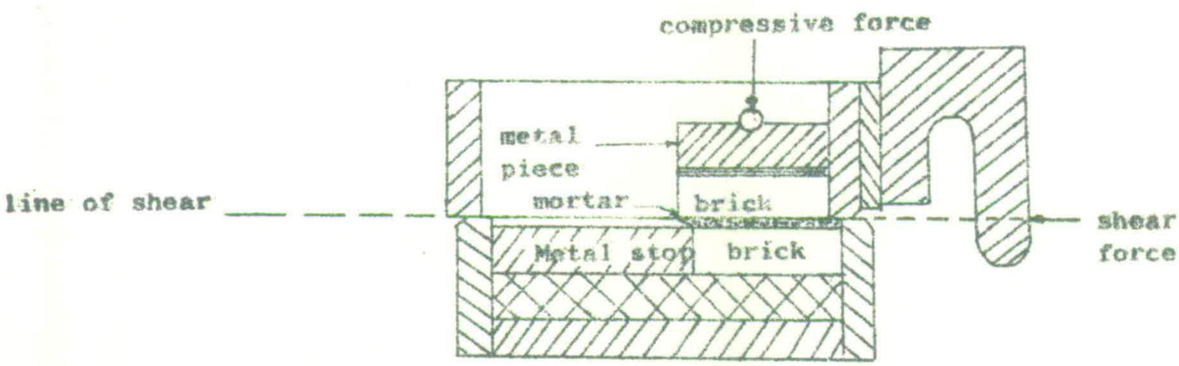


Fig. 5.3 Shear box test arrangement

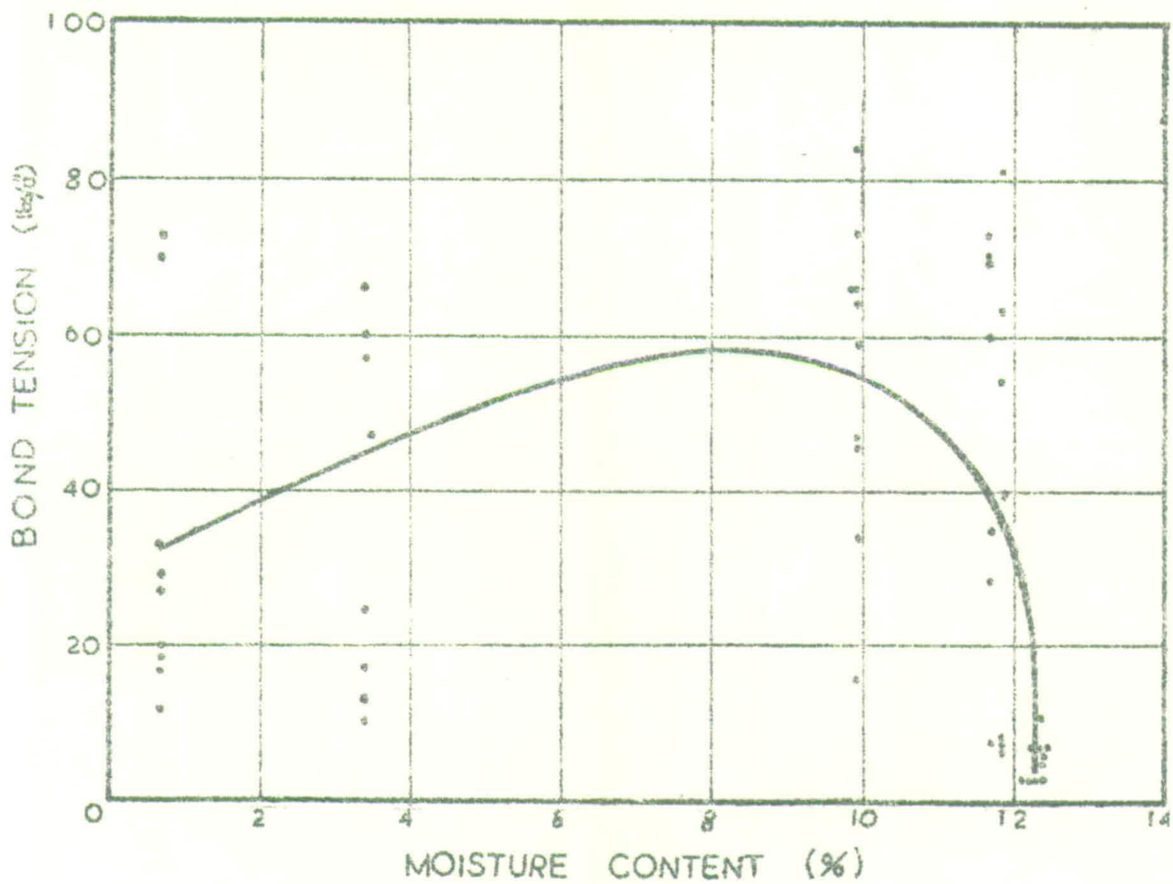


FIG. 5.4 - Relationship between moisture content of brick and bond tension of brick masonry couplets.

agree with the findings of Semenstov, described in Polyakov's work[46] who concluded that the wetting of bricks, before laying with cement mortar substantially increases the bond, but that saturated bricks lead to a large reduction in bond strength.

When dry bricks are laid they absorb water from the layer of mortar in contact with the brick and there may be insufficient water for the hydration of the cement to take place. In this case the strength of the tensile bond, between mortar and brick will be less than optimum.

When saturated bricks are laid, they absorb little or no water from the mortar and generally the excess water in the mortar, over and above that required for the hydration of the cement, will remain and could cause the strength of the tensile bond, mortar to brick, to be less than optimum.

Loads were placed on the couplets of brick masonry during the curing period to represent the load occurring in full-size construction coming from the several courses of brickwork laid above. The maximum applied stress of 8 lb/in^2 is equivalent to that from a full storey height of brickwork. The results of these tests are given in Tables 5.4 (a) and 5.4 (b). From these results there appeared to be no specific relationship between tensile bond strength and applied compression during curing. There was a wide scatter of experimental results indicating the presence of uncontrolled variables such as the surface state of brick and the difference in suction rate at the same moisture content. The results confirm earlier work[39]. Plate 5.2 shows the typical failure of couplet in bond tension.

5.4. BOND SHEAR TESTS

The tests were conducted in a similar soil mechanics shear-box as shown in Fig. 5.3. The couplets were subjected to pure shear at the interface of brick-mortar joint with the complete absence of the compressive force. The results are shown in Tables 5.5 and 5.6.(a+b);P.49,50. Here also the moisture content has a marked effect on the bond shear, but no significant effect was noticed when load was placed on the specimen during the curing period. The results again agree with earlier work[39].

5.5. RELATIONSHIP BETWEEN BOND SHEAR AND BOND TENSION

From the results of the tests and from Figures 5.5 and 5.6, the following general relationship could be put forward for solid bricks and 1:3 cement mortar, between bond tension f_{tb}, and bond shear V_b:

$$V_b = 8.8\, f_{tb}^{\,0.5}$$

Murthy[39] found in his test on the model bricks and 1:3 cement mortar that bond shear was 2.3 times bond tension. Whilst this is true of a particular case, it is not necessarily generally applicable.

Polyakov[46] found that the ratio of bond shear to bond tension depends on the value of bond tension and gave the relationship as follows:

$$\frac{V_b}{f_{tb}} = 2.25 - 0.5\, f_{tb},$$

where $V_b \ngeq 2.5\ \mathrm{kg/cm^2}$.

The/

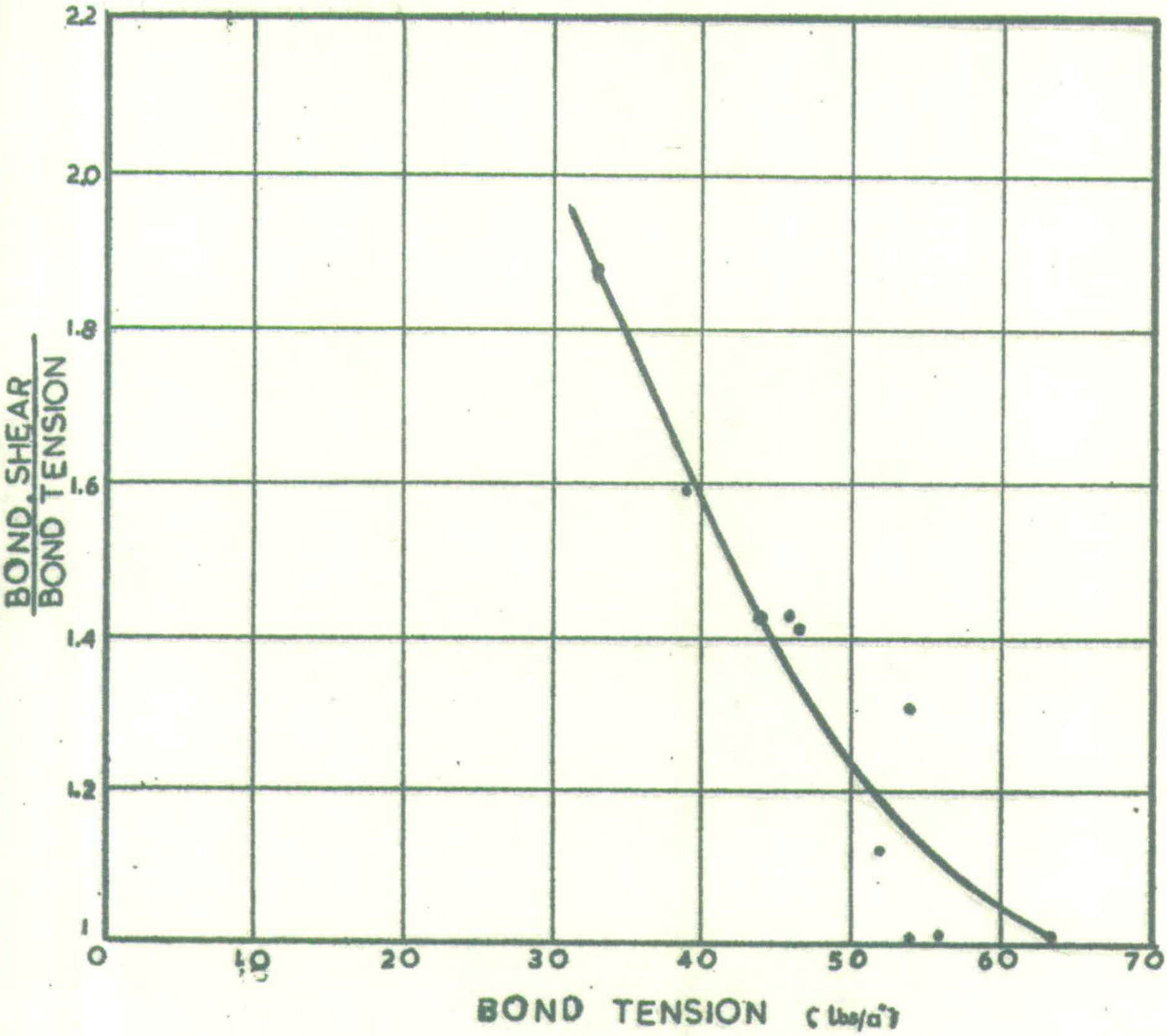


FIG. 5·5 - **Relationship between the bond tension and bond shear of brick masonry couplets.**

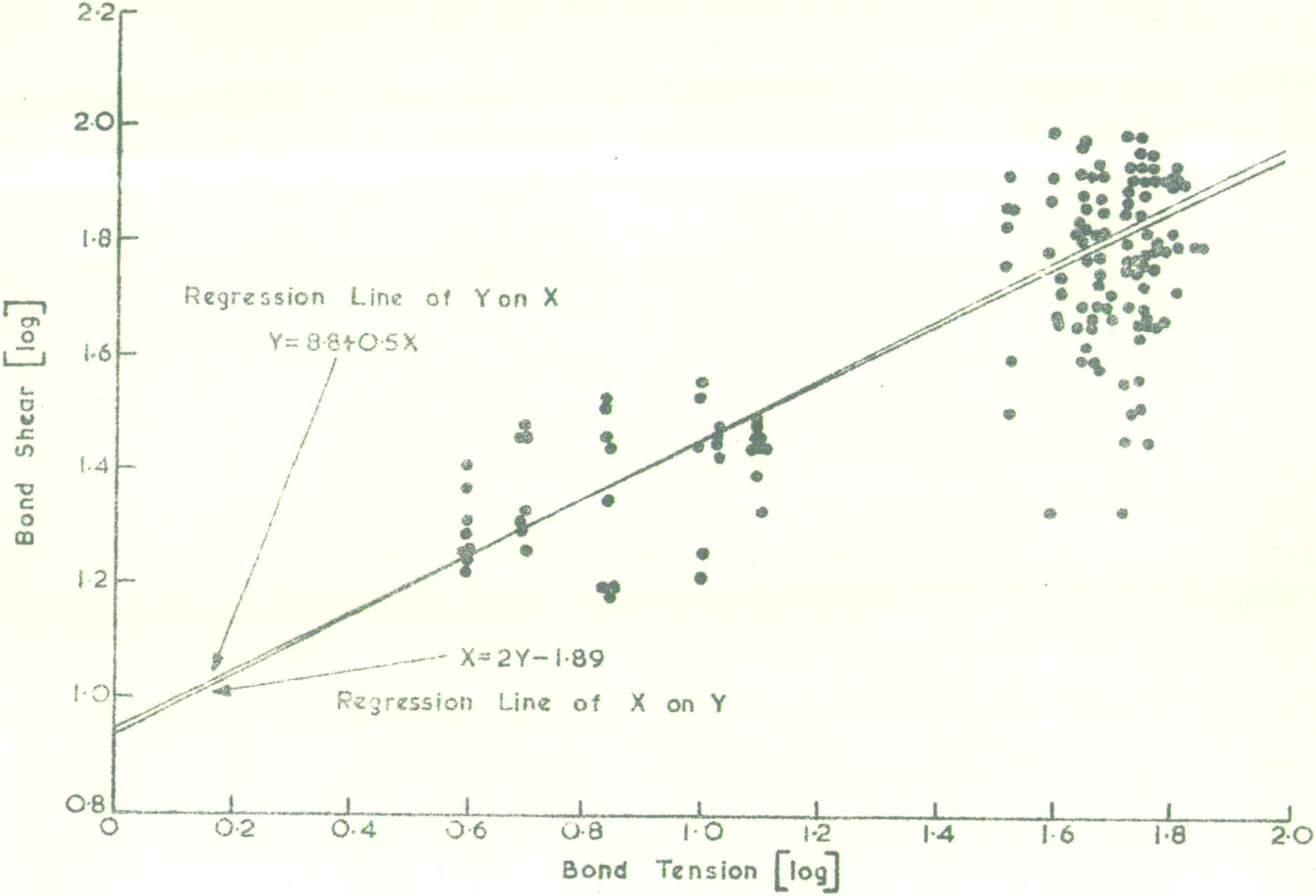


FIG. 5·6

Relationship Between the Bond Tension and Bond Shear of Brick Masonry Couplets

The Russian standards and code have limited the permissible bond tension to 1.8 kg/cm^2 (25.6 lb/in^2) and Semenstov suggested the following relationship: $V_b = 1.7\ f_{tb}$.

The general relationship $V_b = 8.8\ f_{tb}^{0.5}$ agrees reasonably well with those discussed above.

5.6 EFFECT OF PRECOMPRESSION ON SHEAR STRENGTH OF BRICK MASONRY COUPLETS

The shear strength of brick masonry couplets due to friction alone is directly proportional to the normal pressure on the plane of shear provided failure occurs at the interface of brick and mortar. However, in brick masonry couplets, the total shear strength consists of bond shear and frictional resistance and can be represented by $V_b = V_{bo} + f\,\sigma y$,

where V_{bo} = bond shear at zero load on brick mortar boundary,

f = coefficient of friction

V_b = horizontal shear stress on brick mortar boundary

σy = compressive stress normal to shearing interface.

5.6.1 SHEAR STRENGTH DUE TO FRICTION ONLY

The first series of tests were conducted to establish the coefficient of friction and initial bond shear in the absence of compressive load and thereby to predict the strength of couplets with friction and bond shear. The couplets were made by placing tissue paper between the top brick and the mortar joint to eliminate the bond shear. The tests were carried out in the soil mechanics shear-

box/

shear-box after removal of the tissue paper. The compressive load was applied prior to the application of shear load. The relationship between compressive stress and frictional shear stress is linear up to the shear stress of 154 lb/in^2, the compressive stress being 200 lb/in^2 at this limit. At higher compressive stresses the shear stress at failure, remained at 154 lb/in^2 and the mortar bed failed at the interface of the lower brick and mortar completely, (Plate 5.3) making it impossible to find the true frictional resistance because of the sliding of the mortar bed on the lower brick. The results are shown in Table 5.7.P.51. The principal stresses set up in the mortar joints due to the combined compressive stress and shear stress are much below the strength of the mortar in tension or compression. The shear stress was also much below the shear strength of the mortar. The coefficient of friction, found statistically, was 0.74, which was comparable to that found in earlier work[39,40].

The shear strength of the couplets up to 150 lb/in^2, where bond shear has been eliminated could be given by $V_f = f\,\sigma y$ where V_f = shear strength of couplets due to friction alone on brick mortar boundary.

5.6.2. SHEAR STRENGTH DUE TO FRICTION AND BOND

To predict the shear strength, when bond shear was not eliminated, the value of bond shear at zero compressive stress was determined experimentally, and added to the relationship given above. The results, shown in Fig. 5.7 were calculated from the formula:

V_b/

5 - Plate 5.2 - 5.3

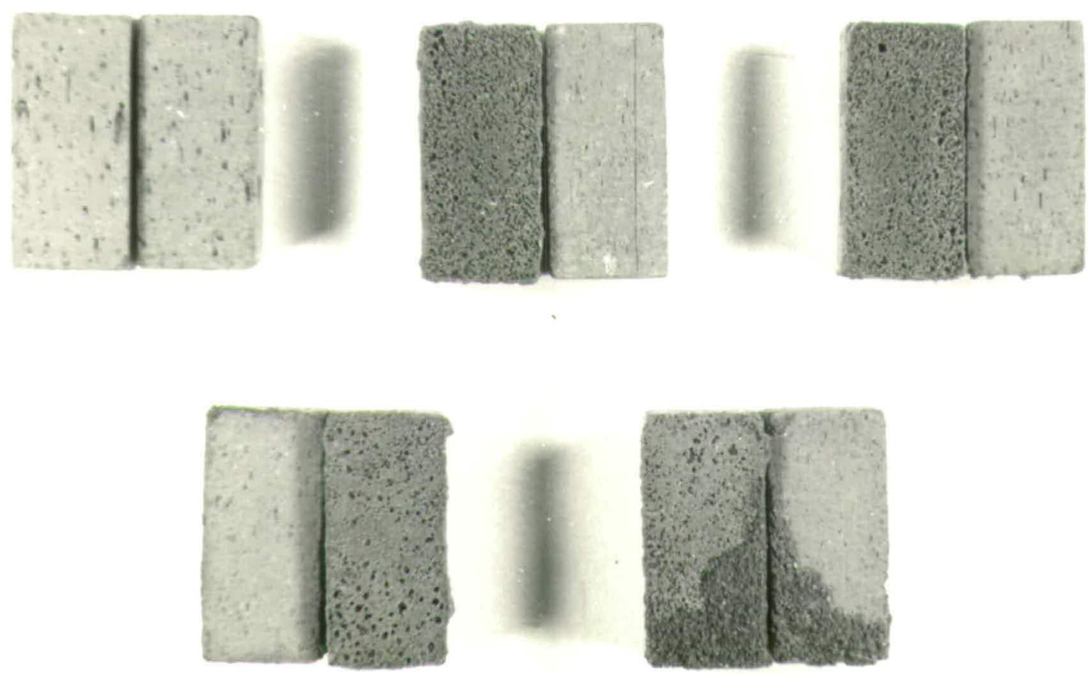


Plate 5.2 - Typical failure of couplets in bond tension test.

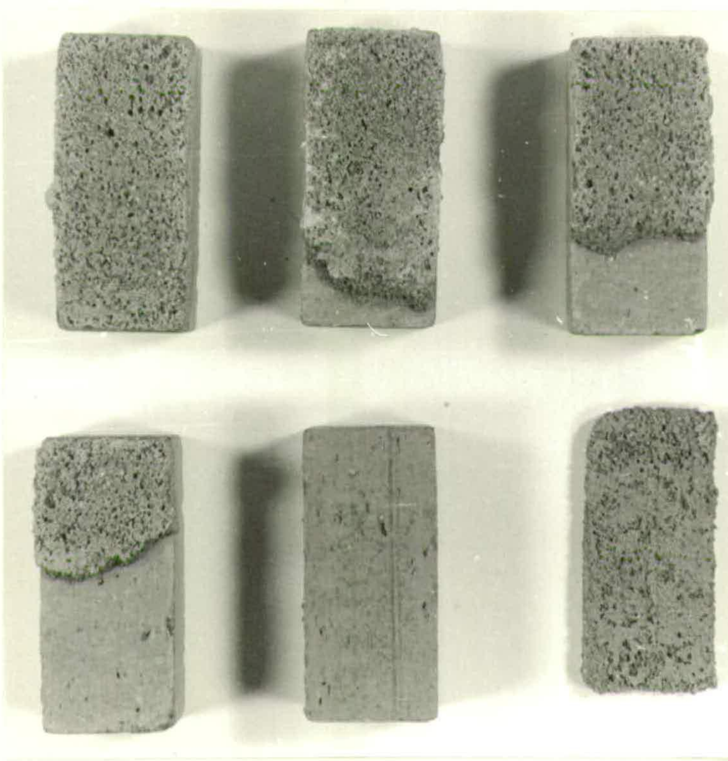


Plate 5.3 - Typical failure stages of couplets in bond shear test.

$$V_b = V_{ho} + f\,\sigma y,$$

where $V_{bo} = 30.6\ \text{lb/in}^2$, (Table 5.8)

$f = 0.74$. (section 5.6;1)

The second series of tests were conducted where bond shear was not eliminated. The object of the tests was to find out the effect of precompression on the initial bond shear and to compare the predicted value with the actual results. The tests were conducted in a shear box and the results are shown in Table 5.8. (P.53.) The relation between the compressive stress and shear stress was found to be linear up to a shear stress of 181 lb/in^2, the compressive stress being again 200 lb/in^2 at this limit. Once this limit is reached, an increase in the compressive stress does not increase the shear strength up to a compressive strength of 245 lb/in^2. There is an apparent increase in shear resistance when the compressive stress exceeds 245 lb/in^2; which is explained in section 6.6., Chapter 6.

Similar phenomena were noticed by BENJAMIN and WILLIAMS[5]. However, above this limit of 181 lb/in^2 shear stress at 200 lb/in^2 compressive stress, the shear strength may be calculated from friction alone.

After the failure of the above couplets, the shear load was withdrawn and the top brick was placed in position and tested again to find out the shear strength due to friction alone. These results are shown in Table 5.7, and it can be seen that these agree very closely with the frictional shear/

shear strength of the couplets where the initial bond shear was eliminated by placing the tissue paper (Figure 5.7).

Up to a shear stress of 181 lb/in^2 the shear strength of the couplet is given by:

$$V_{ult} = V_{bo} + f \sigma y,$$

where f = coefficient of internal friction,

V_{ult} = ultimate shear strength.

It can be seen from Figure 5.7 that the predicted value is quite near to the one found by the first series of tests. Hence, it appears that the coefficient of internal friction is the same as the coefficient of external friction. The initial bond shear appears to be independent of the normal pressure up to the limit given above.

5.7 CONCLUSIONS

1. The bond strength of the model brickwork described in this note, made with 1:3 cement mortar, varies considerably with changes in the moisture content of bricks at the time of laying. For maximum bond strength for a particular mortar there would appear to be an optimum value of moisture content, which in the case of the bricks tested is approximately two-thirds of the water absorption of bricks determined by the 24-h immersion test (by weight) (see Fig. 5.4).

2. The bond shear and bond tension strengths of the couplets were independent of the load placed on them during the curing period.

3. There is a statistical indication of a non-linear correlation between bond/

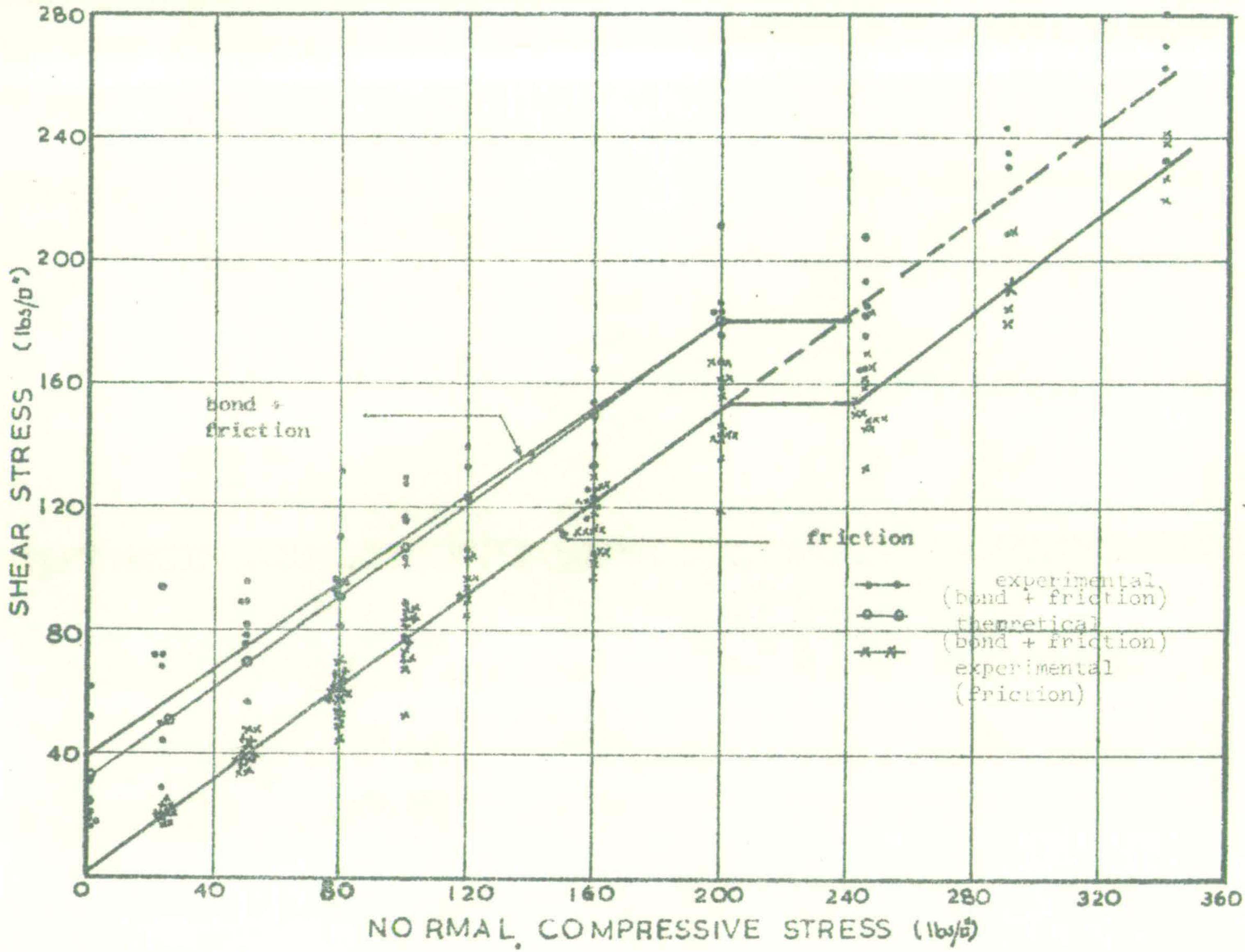


Fig. 5.7 - Shear strength of brick masonry couplets subjected to vertical compression.

bond shear and bond tension (Fig. 5.5 and 5.6).

4. The shear strength of brick masonry couplets due to friction alone is proportional to the normal pressure on the plane of shear, up to a limit. For the model bricks and 1:3 mortar described in this note the limit of normal compressive stress was 200 lb/in^2.

5. The initial bond shear is independent of the normal pressure on the shearing interface up to the limit. The limit depends on the diagonal tensile strength of the brickwork (section 6.6).

6. The value of the coefficient of internal friction is very close to that of external friction.

TABLE 5.1

PHYSICAL PROPERTIES OF BRICKS

Crushing strength (lb/in^2)	4332
Range	3490 - 6005
Standard deviation	624
Coefficient of variation (%)	14
Water absorption:	
wt. % after 24 h	12.65
wt. % after 5 h boiling	13.75
vol. % after 24 h	25.60
vol. % after 5 h boiling	27.60
Saturation coefficient	0.911

TABLE 5.2

COMPRESSIVE STRENGTH TESTS ON 2.78-in. CEMENT MORTAR CUBES

Age at test (days)	Cube No.	Crushing strength (lb/in^2)	Average value (lb/in^2)	Allowable value according to B.S. 12:1958 (lb/in^2)
3	1 2 3	6070 5070 5550	5560	3000
7	1 2 3	7100 5000 7540	6550	4000
28	1 2 3	7650 7400 7540	7530	-

TABLE 5.3

COUPLET BOND TENSION STRENGTH

Area of couplets = 1.0 in^2

	Treatment of brick before use					
	Dry	Dipped in water for 5 s	Dipped in water for 2 min	Dipped in water for 5 min	Dipped in water for 10 min	Dipped in water for 2 h
Mortar strength - 1-in. cube (lb/in^2)		1,900				1,120
Moisture content (%)	0.66	3.38	9.91	11.66	11.81	12.17
Bond tension strength (lb/in^2)	20.38 16.50 11.75 70.00 72.00 29.00 18.00 33.00 27.00	57.25 10.25 60.00 24.75 17.00 13.00 100.00 66.00 48.00	84.88 45.75 15.75 34.00 47.00 59.00 66.00 73.00 64.00 66.00	39.75 28.63 7.50 69.00 60.00 70.00 45.00 35.00	7.50 81.00 63.00 40.00 7,00 64.00 43.00 8.00	2.50 6.90 6.25 2.25 6.00 10.75 6.00 2.25 5.50 2.25
	Av.33.07	44.0	55.54	44.40	39.20	5.06

TABLE 5.4 (a)

EFFECT OF LOAD PLACED ON COUPLETS DURING CURING PERIOD

Area of couplets = 1 in^2

Load (lb.)	0	2	4	8
Bond tension strength (lb/in^2)	2.50	8.50	4.00	7.82
	6.90	5.00	0.00	3.00
	6.25	10.00	6.50	9.94
	2.25	12.90	2.25	
	6.00	5.00	2.25	
	10.75	10.62	2.25	
	6.00	5.00	9.70	
	2.25	27.25		
	5.50	4.75		
	2.25			
	Av.5.06	9.90	3.90	6.92

Mortar strength (1-in. cube) = 1120 lb/in^2 (Average of four cubes).
Moisture content of brick before laying = 12.17%

TABLE 5.4 (b)

EFFECT OF LOAD PLACED ON COUPLETS DURING CURING PERIOD

Area of couplets = 1 in^2

Load (lb.)	0	2	4	6	8
Bond tension strength (lb/in^2)	77.50	56.75	32.25	28.25	109.25
	68.10	41.90	60.0	30.00	75.12
	73.00	58.00	82.10	62.50	41.00
	43.10	30.25	60.25	42.00	57.12
	46.25	30.50	40.75	59.75	29.00
	24.75	85.75	45.50	81.25	50.00
	95.00	49.50	29.00	46.00	38.00
	44.00	74.00	59.00	69.00	48.00
	79.00	63.00	35.00	64.00	36.00
	58.00	62.00	32.00	81.00	64.12
	89.00	52.00	43.00		29.00
	Av.63.40	54.90	47.20	56.40	52.30

Mortar strength (1-in. cube) = 2120 lb/in^2
Moisture content of bricks = 10.41%.

TABLE 5.5

COUPLET BOND SHEAR STRENGTH

Area of couplets = 1 in^2.

	Dry	Dipped in water for 5 s	Dipped in water for 2 min	Dipped in water for 5min	Dipped in water for 10min	Dipped in water for 2h
Mortar strength - 1-in. cubes (lb/in^2)	1900					1120
Moisture content (%)	0.66	3.38	9.91	11.66	11.81	12.17
Bond shear strength lb/in^2.	32.40	50.40	64.80	45.72	56.88	28.70
	39.60	46.80	46.80	39.60	46.80	20.00
	59.40	57.60	46.80	39.60	46.80	12.95
	68.40	46.08	50.40	43.20	49.32	21.30
	83.60	97.20	94.40	88.20	21.60	28.80
	75.60	86.40	86.40	75.60	108.00	18.00
	75.60	64.80	109.60	61.20	86.40	9.00
		86.40	84.80	97.20	52.40	30.06
			46.80	79.20	75.60	
			84.80		75.60	
Av.	62.10	67.12	71.60	63.30	61.94	21.10

TABLE 5.6 (a)

EFFECT OF LOAD PLACED ON COUPLETS DURING CURING PERIOD.

Area of couplets = 1 in^2

Load (lb)	0	2	4	8
Bond shear strength (lb/in^2)	28.70	7.20	19.85	15.20
	20.00	16.20	20.60	45.70
	12.95	18.00	18.00	27.70
	21.30	33.50	17.30	16.90
	28.80	36.00	18.00	16.90
	18.00	36.00	13.70	28.80
	9.00		10.10	34.20
	30.06			33.10
				22.70
	Av.21.10	23.30	16.80	24.90

Mortar strength (1-in. cubes) = 1120 lb/in^2

Moisture content of bricks = 12.17%

TABLE 5.6 (b)

EFFECT OF LOAD PLACED ON COUPLETS DURING CURING PERIOD

Area of couplets = 1 in^2

Load (lb.)	0	2	4	6	8
Bond shear strength lb/in^2	54.00	86.40	50.40	28.80	21.60
	82.80	36.00	61.20	68.40	36.00
	64.80	64.80	79.20	79.20	72.00
	64.80	32.40	39.60	86.40	50.40
	64.80	82.80	57.60	61.20	57.60
	36.00	93.60	90.00	63.00	100.08
	86.40	32.40	74.88	64.80	64.80
	82.40	48.24	86.40	64.80	57.96
	64.80	57.60	66.60	54.00	79.20
		43.20	72.00	57.60	28.80
			54.00	46.80	73.80
	Av.66.80	57.74	66.50	61.40	58.40

Moisture content of brick = 10.41%

Mortar strength (1-in. cubes) = 2120 lb/in^2.

TABLE 5.7

SHEAR STRENGTH OF COUPLETS OF BRICK MASONRY SUBJECTED TO VERTICAL COMPRESSION

Test No.	Normal compressive stress (lb/in^2)	Ultimate shear where bond shear was eliminated by tissue paper (lb/in^2)	Coefficient of friction	Ultimate shear after bond shead has been eliminated (lb/in^2)	Coefficient of friction
1		23.76		21.60	
2		21.60		20.51	
3		20.40		19.80	
4	25.0	25.20	0.858	23.12	0.856
5		21.20		20.16	
6		21.60		23.12	
7		19.80		21.80	
Average		21.44		21.40	
8		39.60		39.60	
9		41.40		36.72	
10		38.90		46.44	
11	50	45.00	0.80	42.20	0.84
12		38.90		42.92	
13		38.90		42.92	
14		36.72		42.20	
Average		39.90		42.00	
15		59.70		59.40	
16		63.00		57.60	
17		65.50		61.20	
18		46.10		49.68	
19	[illegible]	61.60	0.735	61.20	0.74
20		58.60		70.20	
21				52.20	
22				64.80	
Average		59.10		59.76	
23		53.30		86.04	
24		84.00		75.60	
25	100	67.40	0.788	82.44	0.776
26		103.00		68.40	
27		88.50		71.28	
28		76.60		77.04	
28a				82.44	
Average		78.80		77.60	
29		97.10		97.10	
30		91.00		90.00	
31	120	105.00	0.805	86.00	0.786
32		93.50		104.50	
Average		96.60		94.40	

(contd.)

TABLE 5.7 (continued)

Test No.	Normal compressive stress (lb/in^2)	Ultimate shear where bond shear was eliminated by tissue paper (lb/in^2)	Coefficient of friction	Ultimate shear after bend shear has been eliminated (lb/in^2)	Coefficient of friction
33		112.00		129.60	
34		128.00		108.00	
35		115.00		115.00	
36		122.00		108.00	
37	160	119.00	0.72	124.00	0.718
38		115.00		126.00	
39		121.00		104.50	
40		97.50		102.50	
41		122.00			
Average		116.70		114.70	
42		143.64		147.80	
43		168.4[illegible]		158.50	
44		137.00		144.00	
45	200	119.00	0.725	163.00	0.784
46		146.00		162.00	
47				155.00	
48				168.00	
Average		145.00		156.90	
49		158.50		165.60	
50		191.00		162.[illegible]0	
51		151.00		151.20	
52	245	133.00	0.638	147.60	0.64
53				147.60	
54				173.00	
55				151.00	
Average		156.00		156.90	
56		191.00			
57	291	185.00	0.654		
58		210.00			
59		180.00			
Average		191.50			
60		23.90			
61	341	241.00	0.680		
62		227.00			
63		220.00			
Average		232.00			

Area of couplets = 1 in.^{2}. Mortar strength (1-in. cubes) = 1926 lb/in^2.
Tensile strength, (standard briquette) = 230 lb/in^2.
Shear strength (1-in. cube) = 291 lb/in^2 (precompression 50 lb).
Moisture content of brick = 12.19%.

TABLE 5.8

SHEAR STRENGTH OF COUPLETS OF BRICKS SUBJECTED TO VERTICAL COMPRESSION WHERE BOND SHEAR WAS NOT ELIMINATED

Test No.	Normal compressive load (lb/in^2)	Ultimate shear stress (lb/in^2)
1		32.40
2		21.60
3		16.32
4		61.92
5	0	52.20
6		17.28
7		18.00
8		25.22
Average		30.62
9		94.40
10		75.60
11		28.80
12	25.0	68.40
13		43.20
14		50.40
15		75.50
Average		62.30
16		96.48
17		75.24
18		90.00
19		79.20
20	50	82.44
21		90.00
22		57.50
Average		84.40
23		111.60
24		90.00
25		95.40
26		61.20
27	80	82.80
28		93.60
29		132.40
30		93.60
Average		95.10
31		118.80
32		130.44
33		117.44
34	100	79.20
35		89.64
36		129.60
37		104.40
Average		109.90
38		140.40
39		136.08
40	120	106.20
41		122.40
Average		126.77
42		154.80
43		135.00
44		165.50
45	160	105.00
46		126.00
47		140.00
48		162.00
49		124.00
50		115.00
Average		136.40
51		160.00
52		184.00
53		167.00
54	200	176.00
55		186.00
56		184.00
57		212.00
Average		181.30
58		194.50
59		187.00
60		176.50
61	245	165.00
62		165.00
63		183.60
64		208.08
Average		181.09
65		230.40
66	291	237.00
67		209.00
68		245.00
Average		230.00
69		23.20
70	341	270.00
71		281.00
72		263.00
Average		261.50

Area of couplets = 1 $in.^2$ Mortar strength (1-in. cubes) = 1926 lb/in^2.

Mortar shear strength at 50 lb. precompression (1-in. cubes) = 291 lb/in^2.

Mortar tensile strength = 230 lb/in^2 (standard briquettes).

Moisture content of brick = 12.1%.

CHAPTER 6

Shear Tests on Storey-Height Shear-wall Structures with openings, subjected to precompression

6.1 INTRODUCTION

In recent years, there has been increasing interest in load-bearing brickwork walls for non-framed cross-wall type of construction. In a multi-storey building of this type, to safeguard against the 'pack of cards' collapse, shear walls are provided at right angles to the main walls to resist lateral loads due to wind, blast or seismic action. Very little work has been done on the behaviour of shear walls of this type subjected to wind loading. Openings are also provided in the shear walls for various purposes and due to the tendency in the past to avoid the structural use of walls containing openings, little attention has been paid to the investigation of the structural behaviour of such walls.

The primary object of this work was to investigate the structural behaviour and the shear strength of a single-storey shear-wall structure containing door openings and stiffened by cross-walls. Some exploratory tests were carried out on model brickwork couplets, and a relationship established[56] between the normal compressive stres and shear stress. These couplet tests, however, did not represent very closely the effect of wind load on shear walls, since in the couplet tests no shear stress was developed in the brick due to the application of shear load along the mortar joint. In an actual structure, the presence of shear stress both in the brick and mortar will give rise to diagonal tension and compression in the masonry as a whole. Hence, in the test described here, the model structure was subjected to precompression in the vertical direction to simulate/

simulate the actual loading condition in a building before being subjected to the racking load. An effort was made to compare the tests results with the couplet formula to find a suitable tool to predict the shear strength of such structure in advance.

6.2 MATERIALS AND CONSTRUCTION DETAILS

One-sixth-scale model bricks with an average crushing strength of 4332 lb/in^2 and 4221 lb/in^2 were used in the construction of the single-leaf walls, representing a thickness of $4\frac{1}{2}$-in. at full scale. The average water absorption (wt.%) according to the 24-h test was 12.65 for all test structures.

6.2.2. CEMENT AND SAND

The sand and cement were the same as described in Section 5.2.2. and 5.2.3.

6.2.3. MORTAR

1:4 cement and sand mortar by wt. (1:3 by vol.) was used for the construction of the wall. The average crushing strength of the 1-in. mortar cubes for shear walls in different structures varied from 1500 to 2234 lb/in^2. Details are given in Table 6.1.

A mortar mix of 1:1 (cement: sand) was used for assembling walls together and joining them to the slab. The average crushing strength of the cubes (1:1) at the time of testing the structures was 1120 lb/in^2. The structures were tested on the third day after their assembly.

6.2.4. CONSTRUCTION DETAILS

The wall panels were initially built vertically in jigs, as shown in/

in plate 3.2 (Chapter 3) and then assembled according to the layout shown in Figure 6.1. While assembling, care was taken to see that the wall remained plumb and level. The joints between panels, as well as those between the slab and the panels were completely filled with cement mortar. Also the gap in the channel receiving the panel was filled with mortar.

6.3. TESTING EQUIPMENT

6.3.1 LOADING FRAME

The storey-height structure was assembled on a $1\frac{1}{4}$-in. steel receiving channel seated on a frame 8-ft. long, 3-ft. wide made of 4 x 2-in. channel and 4 x $1\frac{3}{4}$-in. I sections. The vertical members of the frame consisted of 4 x $1\frac{3}{4}$-in. I sections. The frame was specially designed to test one-sixth-scale cross-wall structures up to three storeys high. The frame was capable of applying horizontal loadings of up to 10,000 lb. to a model structure.

6.3.2. METHOD OF APPLYING LOAD

The vertical load was applied to the structure by means of lead billets and concrete slabs as shown in Plates 6.2 to 6.6. The racking load was applied by a 6-ton hydraulic jack seated on a semi-circular hinge at the centre of the loading beam. The beam was 1 x $1\frac{1}{4}$ x 10-in. long of high-tensile steel, and was supported on $\frac{1}{2}$-in. -dia. rollers spaced 9 in. apart. The rollers transmitted the load to the slab, through the channel embedded at the edge of the slab along the entire width of the slab as shown in Fig. 6.2.

6.3.3./

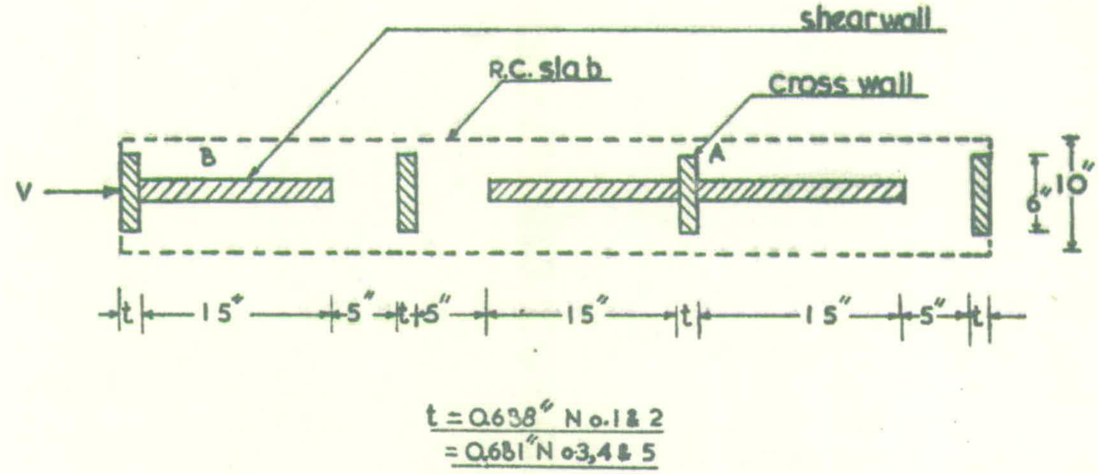


Fig. 6 1. Layout of the storey height test structures. with opening.

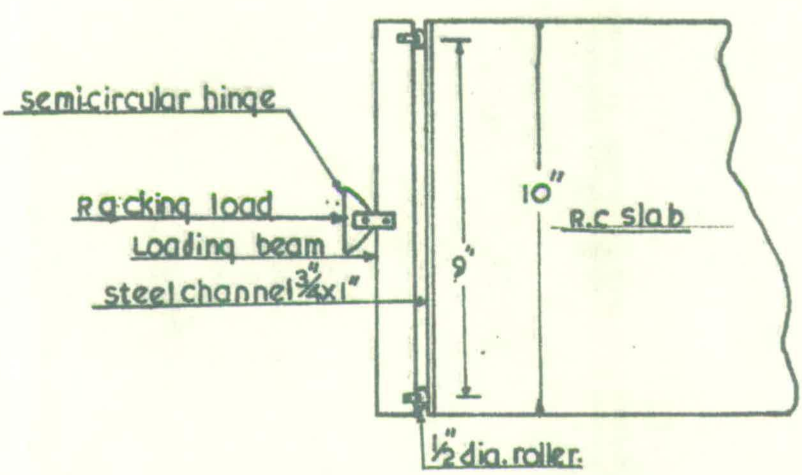


Fig. 6. 2. Arrangement for applying racking load.

6.3.3. Load-measuring Apparatus

The racking load was measured by a special apparatus of approximately 6000-lb capacity. A reference beam ($\frac{1}{4}$ x $\frac{3}{4}$ x 10-in.) for measuring the load was connected by two pins 9-in. apart to the loading beam as shown in 6.1 (Plate). A dial gauge was then mounted on the reference beam to measure the central deflection of the loading beam due to the applied load. The apparatus was calibrated in an Avery Universal testing machine and the calibration curve is shown in Figure 6.3.

6.4. EXPERIMENTAL INVESTIGATION

6.4.1. TEST RESULTS

Five model structures with door openings, subjected to varying precompression, were tested to failure under a racking load. The horizontal deflection at the slab level was measured at regular intervals during loading till failure. The five structures all failed in the same manner with cracks passing through the horizontal and vertical mortar joints. Plates 6.2-5 indicate the typical form of damage at failure. In almost all cases first failure was noticed on the left of the panel 'A' (see Fig. 6.1) and was then followed by cracks in panel 'B', which resulted in the ultimate failure of the structure. A summary of the test results is shown in Table 6.1 (P. 69) and 6.2 (P. 70). Figures 6.4 and 6.5 show the relationship between the racking load, horizontal deflection and shear modulus.

6.4.2./

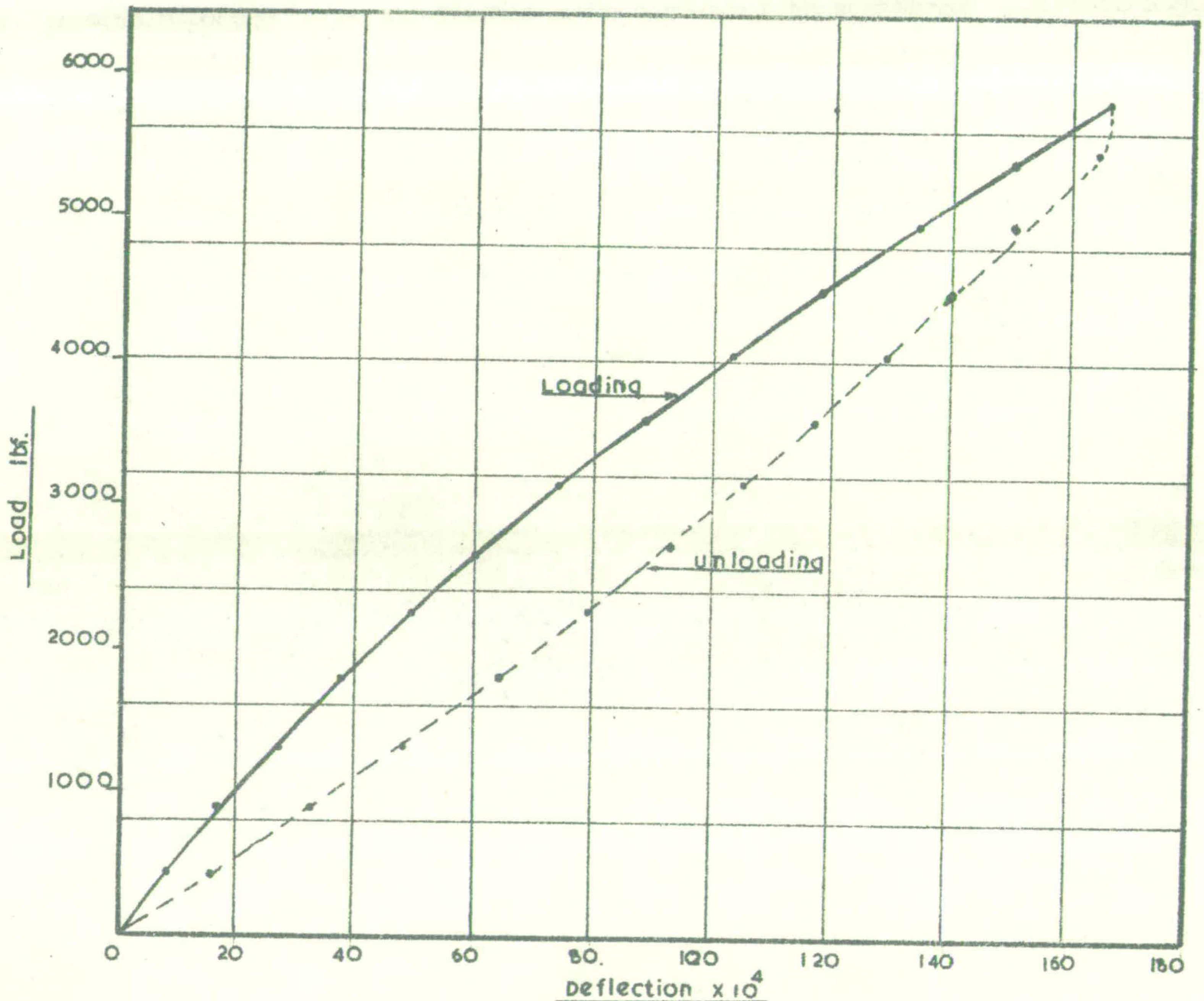


Fig. 6.3. Calibration curve.

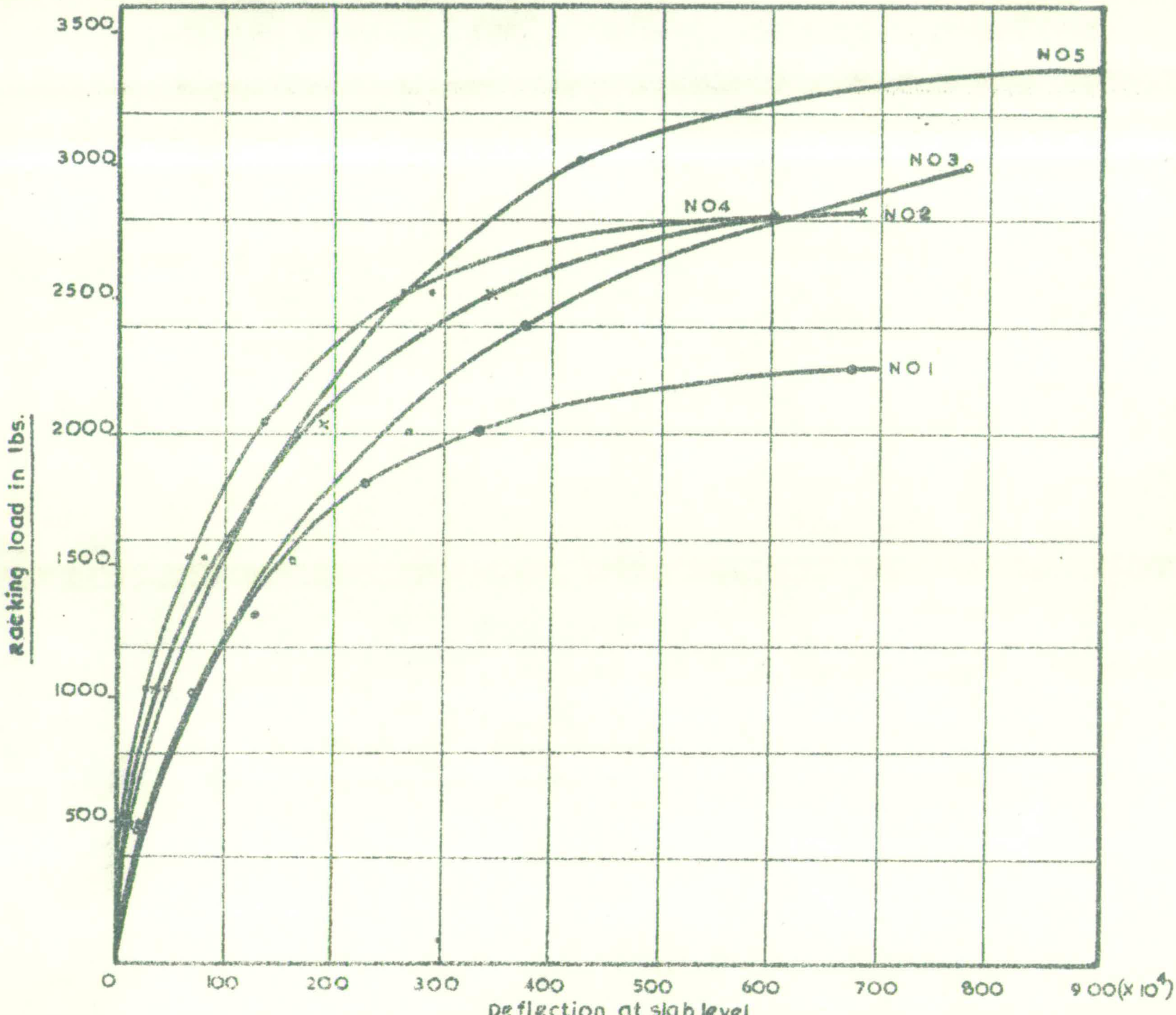


Fig. 6. 4. Relationship between the Racking load and horizontal deflections at slab level

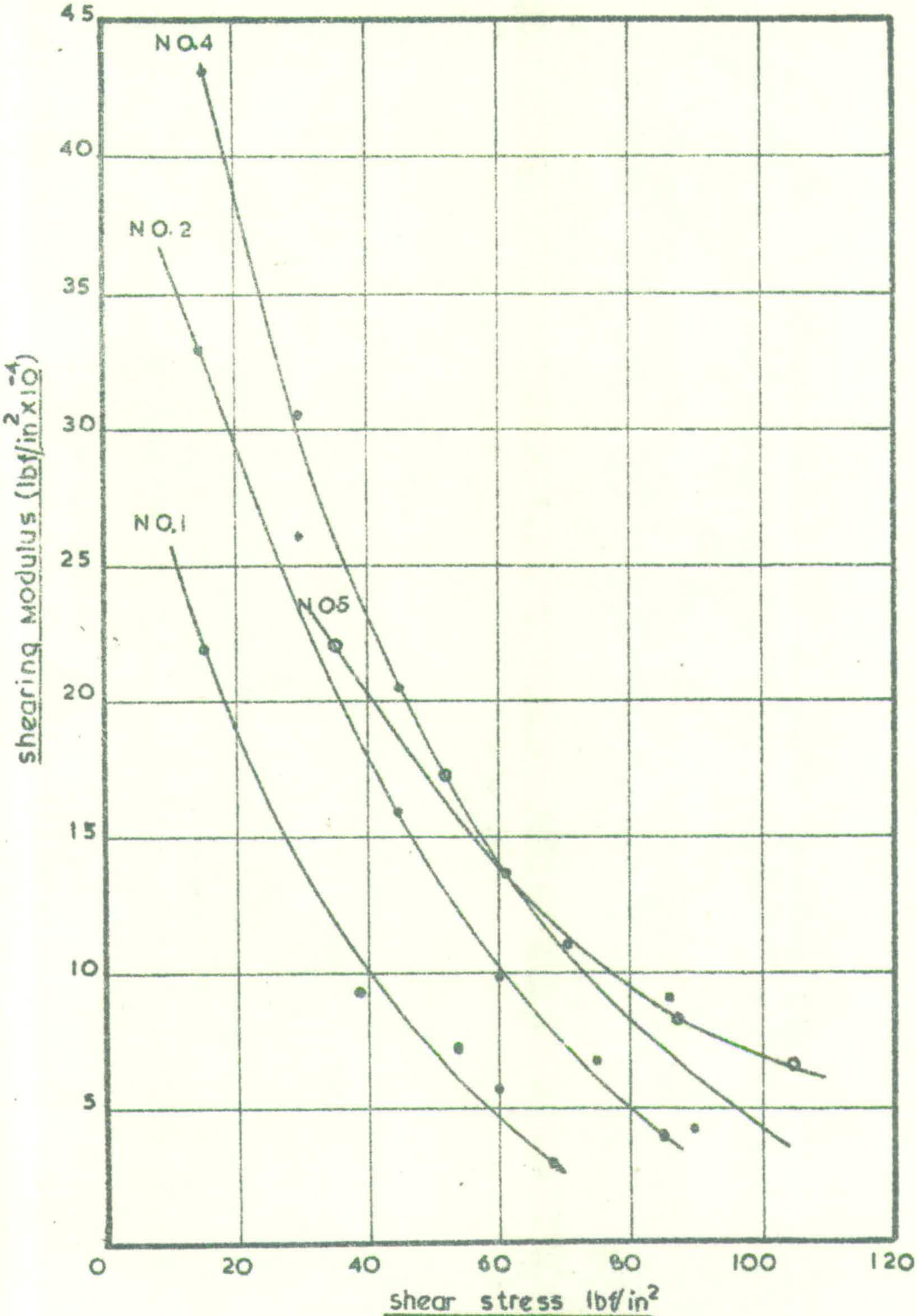


Fig. 6.5. Relationship between the shear stress and shearing modulus.

6 - Plate 6.1 - 6.2

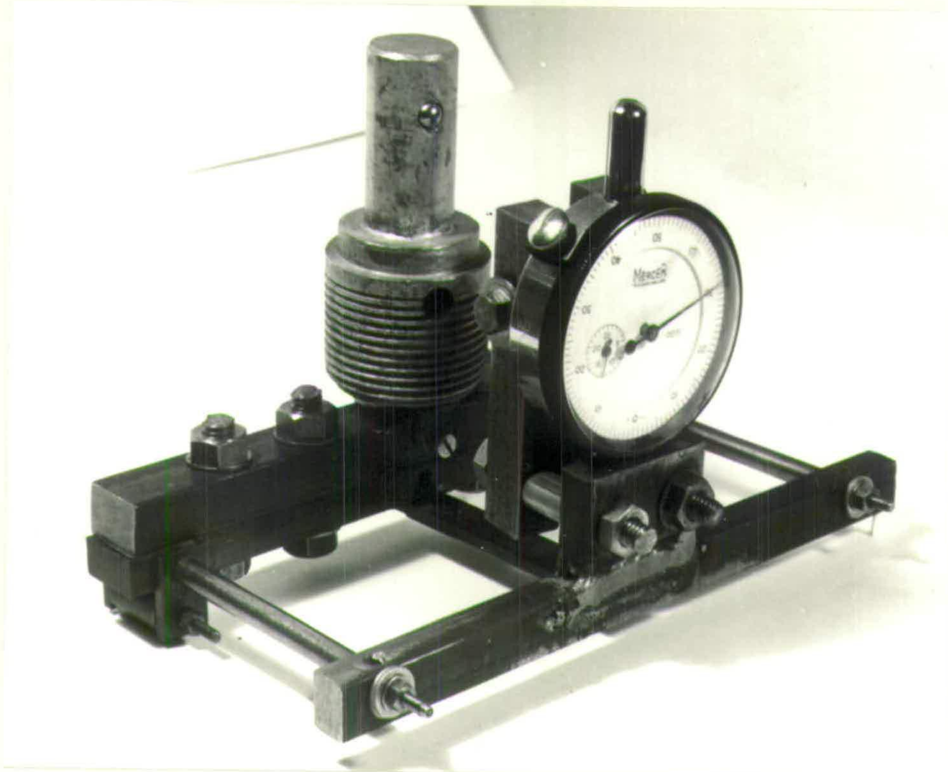


Plate 6.1 - Load measuring beam.

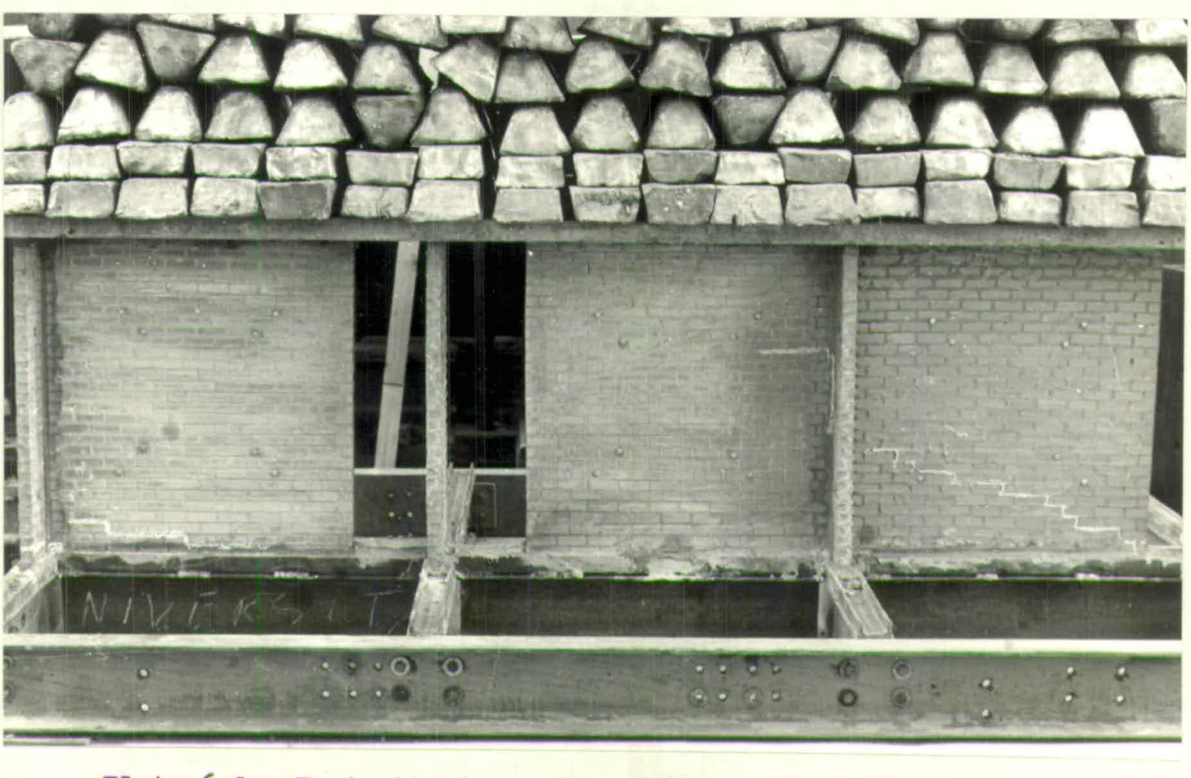
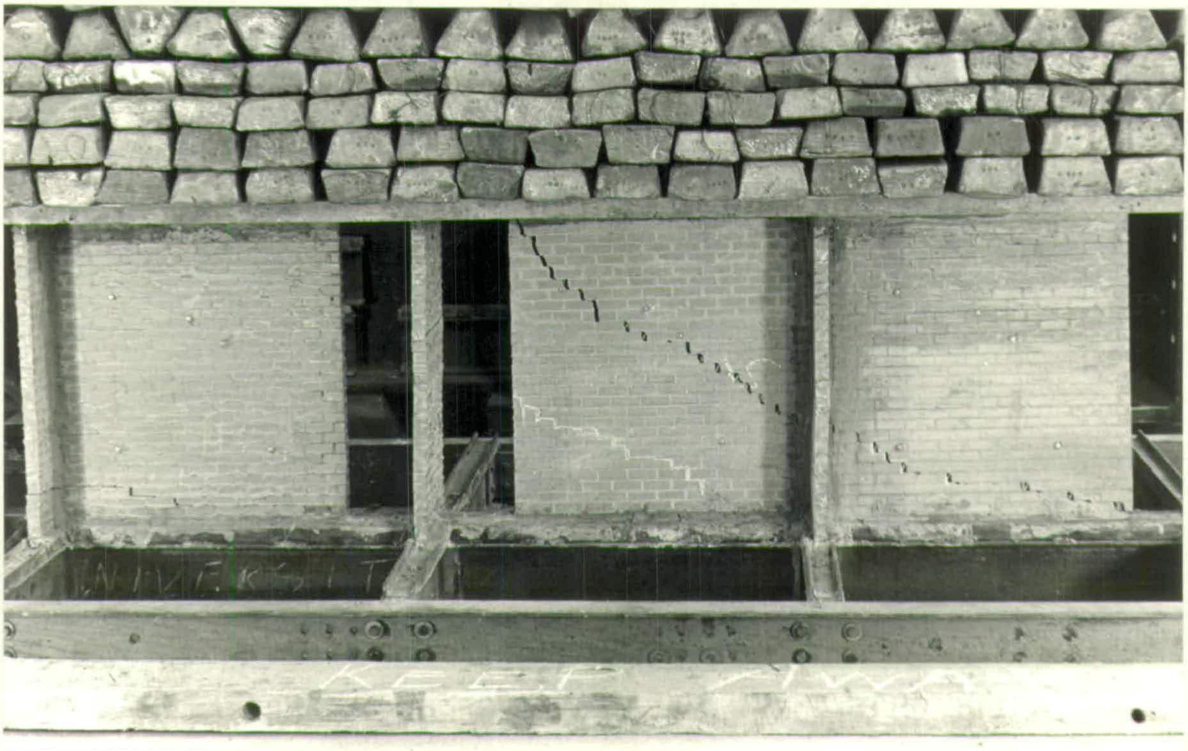
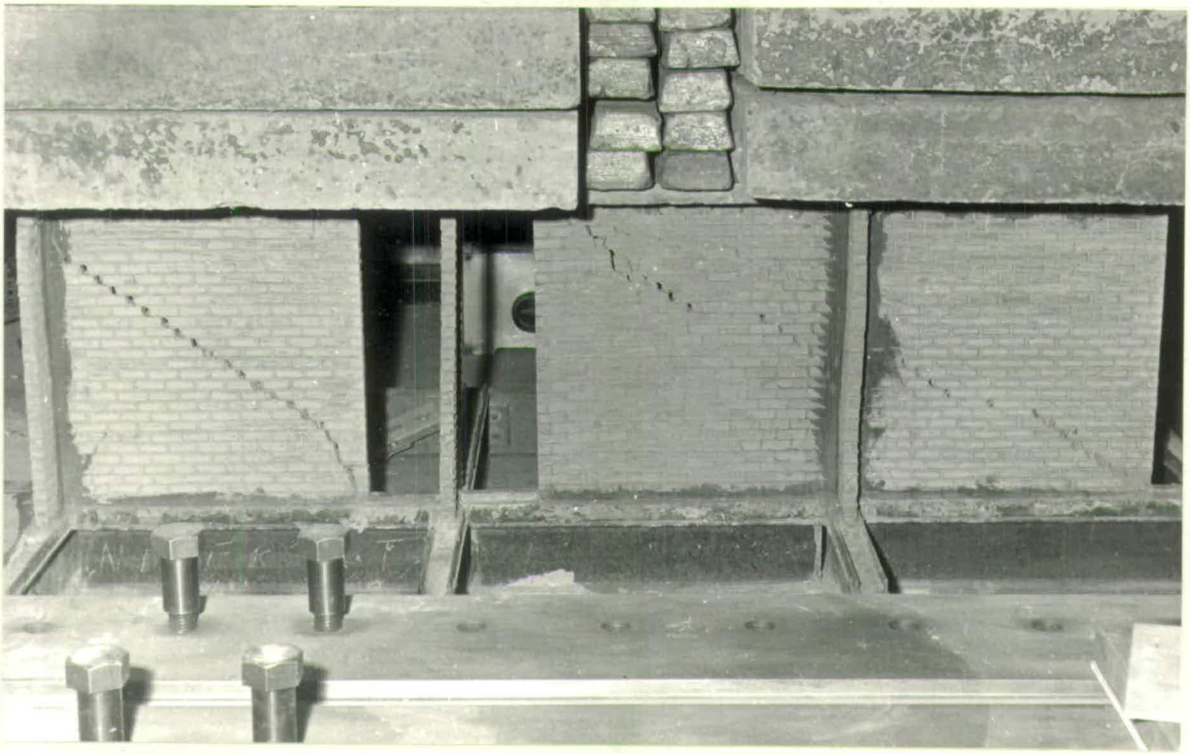


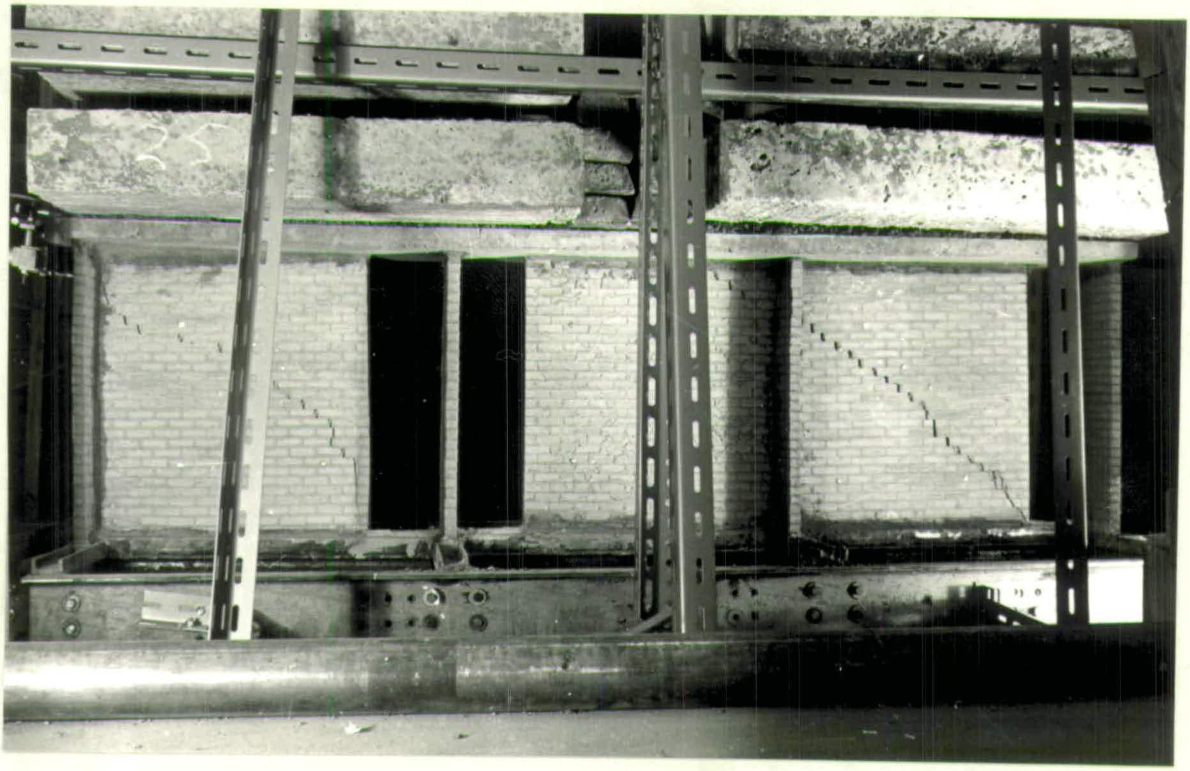
Plate 6.2 - Test structure after failure in test No.1.



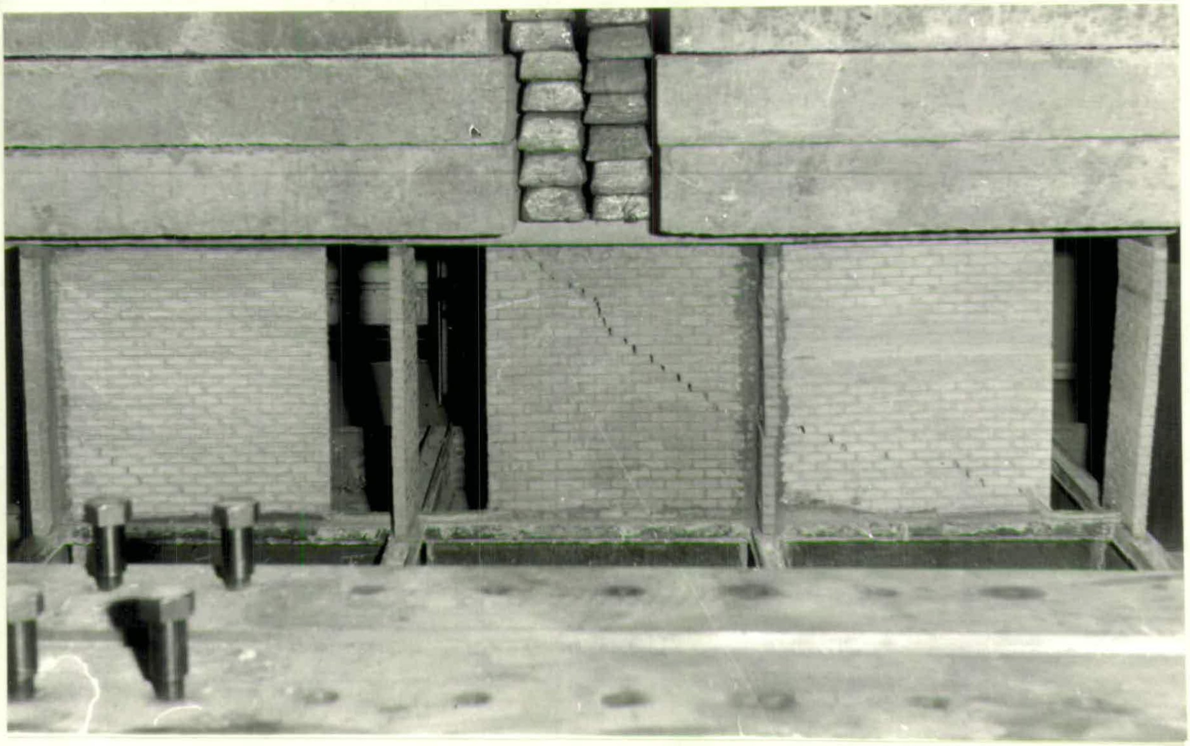
6.3 - Test structure after failure in Test No.2



6.4 - Test structure after failure in test No.3.



6.5 - Test structure after failure in test No.4.



6.6 - Test structure after failure in test No.5.

6.4.2. CALCULATION FROM EXPERIMENTAL RESULTS

In the calculations the following assumptions were made:

(a) The wall foundation provided complete fixity.

(b) The diaphragm was rigid in comparison to the wall.

(c) The lateral deflection due to vertical compression was negligible.

(d) The rigidity of the wall in a direction at right angles to the plane of loading was negligible.

(e) Shear forces caused by external loadings were divided among the walls in proportion to their rigidity.

From the elementary strength of materials, it may be stated that the walls act as deep beams and deform because of bending and shear.

The deflection of the walls (in.):

$$\Delta_1 = \frac{V_1 h^3}{3EI_1} + \frac{1{:}2\ V_1 h}{A_1 G} \qquad \text{.............} \ (1)$$

$$\Delta_2 = \frac{V_2 h^3}{3EI_2} + \frac{1.2\ V_2 h}{A_2 G} \qquad \text{.............} \ (2)$$

The racking load (lb):

$$V = V_1 + V_2 \qquad \text{.............}(3)$$

Assume Poisson's ratio, m = 0.1

The modulus of elasticity (lb/in^2):

$$E = 2\ (1 + m)\ G = 2.2G \qquad \text{.............} \ (4)$$

where G = shear modulus (lb/in^2).

For continuity of the structure

$$\Delta = \Delta_1 = \Delta_2 \qquad \text{.............}(5)$$

From/

From eqns. (1), (2) and (3):

$$\frac{V_1}{V_2} = \frac{\dfrac{h^2}{3I_2} + \dfrac{2.64}{A_2}}{\dfrac{h^2}{3I_1} + \dfrac{2.64}{A_1}} = \frac{R_1}{R_2} \qquad \text{.............(6)}$$

$$\frac{V}{V_2} = \frac{R}{R_2} \qquad \text{.............(7)}$$

where h = height of structure (in.)

A = area of panel (in^2),

I = moment of inertia (in^4),

R = rigidity (lb/in^2)

By knowing h, A_1, A_2, I_1 and I_2, we can calculate V_2 and V_1.

Shear stress in the panels:

$$'A' = \frac{V_1}{A_1} \quad ; \qquad 'B' = \frac{V_2}{A_2}$$

6.5. FORMULA FOR PREDICTING SHEAR STRENGTH OF BRICKWORK

Brickwork subjected to combined stress exhibits two distinct types of failure· (a) by cracking through bricks and mortar, governed by the constant maximum tensile strain or stress, (b) by shear failure at the interface, governed by the shear strength, which consists of initial bond shear and the resistance, proportional to the normal stress, due to friction between brick and mortar.

The diagonal tensile strength of the brickwork was estimated on the basis of experimental work carried out in America[42,61] as described below. It is well known that a circular specimen of brittle material when/

when loaded along the vertical diameter fails in tension; the tensile stress across the horizontal diameter being given by:

$$f_t = \frac{2P}{\pi D t}$$

where f_t = diagonal tension (lb/in^2),

P = load at rupture (lb),

D = specimen dia. (in.),

t = specimen thickness (in.).

If the circular specimen is of brickwork, oriented as shown in Fig. 6.7, the splitting failure gives a measure of the diagonal tensile strength of the brickwork. The results have been reported[42] of a large number of such tests on 15-in.-dia. circular discs of brickwork and the diagonal tensile strength has been correlated with the compressive strength of 16-in.-high brickwork prisms of the same thickness. These results showed that

$$f_t = K\sqrt{f_m}$$

where $K \geq 2$ ratio of splitting

f_m = compressive strength of prism (lb/in^2).

Similar tests on six-course-high prisms of the model brickwork used in the shear panel tests were carried out to find the average compressive strength f_m. Assuming the splitting ratio of 2 to hold good, the diagonal tensile strength of this model brickwork was found to be:

$$f_t = 2\sqrt{1814} = 85\ lb/in^2$$

6.6/

6.6. THE MAXIMUM STRESS THEORY

If it is assumed that failure is determined at a certain stage by the criterion of maximum tensile stress, then:

$$f_t = \sqrt{\frac{\sigma y^2}{4} + \tau^2} - \frac{\sigma y}{2} = \text{constant} \quad \ldots\ldots\ldots\ (8)$$

$$\text{for failure } \tau \geq f\,\sigma y \quad \ldots\ldots\ldots\ (9)$$

where σy = precompression (lb/in^2),

τ = shear stress (lb/in^2).

We assume that the condition of the equation (9) will be fulfilled by two values σy_1 and σy_2 of σy.

We can re-write equation (8) for the two conditions of equation (9):

$$f_t = \sqrt{\frac{\sigma {y_1}^2}{4} + (V_{bo} + f\,\sigma y_1)^2} - \frac{\sigma y_1}{2} \quad \ldots\ldots\ldots\ (10)$$

as $\tau = V_{bo} + f\sigma y$ (couplet formula)

where v_{bo} = initial bond shear (lb/in^2)

$$f_t = \sqrt{\frac{\sigma {y_2}^2}{4} + (f\,\sigma y_2)^2} - \frac{\sigma y_2}{2} \quad \ldots\ldots\ldots\ (11)$$

where $\tau = f\,\sigma y$

If f_t, V_{bo} and f are known we can calculate σy_1 and σy_2.

At the precompressive stresses σy_1 and σy_2 the transitional phase of the failure starts and within these limits failure of the structure or the brickwork couplets will occur by the attainment of the maximum tensile strength.

Below/

Below and above this range, failure will be governed by the shear at the interface. Precompression above σy_2 will suppress the inherent failure due to diagonal tension and modify its value because the structure will take load till friction is overcome (Fig.6.6). Eventually the ultimate shear stress at the interface of brick and mortar, will be limited by the compressive strength of the brickwork.

Hence, ultimate shear may be calculated from the following formulae.

$$V_b = V_{bo} + f\sigma y \qquad \text{.... (12)} \quad \sigma y \leq \sigma y_1$$

$$f_t = \sqrt{\frac{\sigma y^2}{4} + V_b^{\,2}} - \frac{\sigma y}{2} \qquad \text{.... (13)} \quad \sigma y \geq \sigma y_1 \leq \sigma y_2$$

$$V_b = f\sigma y \qquad \text{.... (14)} \quad \sigma y \geqslant \sigma y_2 \text{ compressive strength of brickwork.}$$

6.7 DISCUSSION OF THE RESULTS

The shear strength of the brickwork was calculated from the above formulae and compared in Fig. 6.6 with the test results on couplets and the model structures Fig. 4.6 by taking $f_t = 85$ lb/in^2 (section 6.5), $f = 0.74$ and $V_{bo} = 40$. (section 5.6.1 and Fig.5.7).

The ultimate shear stresses calculated from the suggested formulae (Fig. 6.6) are in good agreement with the experimental results of the full-size brickwork obtained from other sources[46,53].

Practically no increase in the shear strength of the structure was noticed[56]/

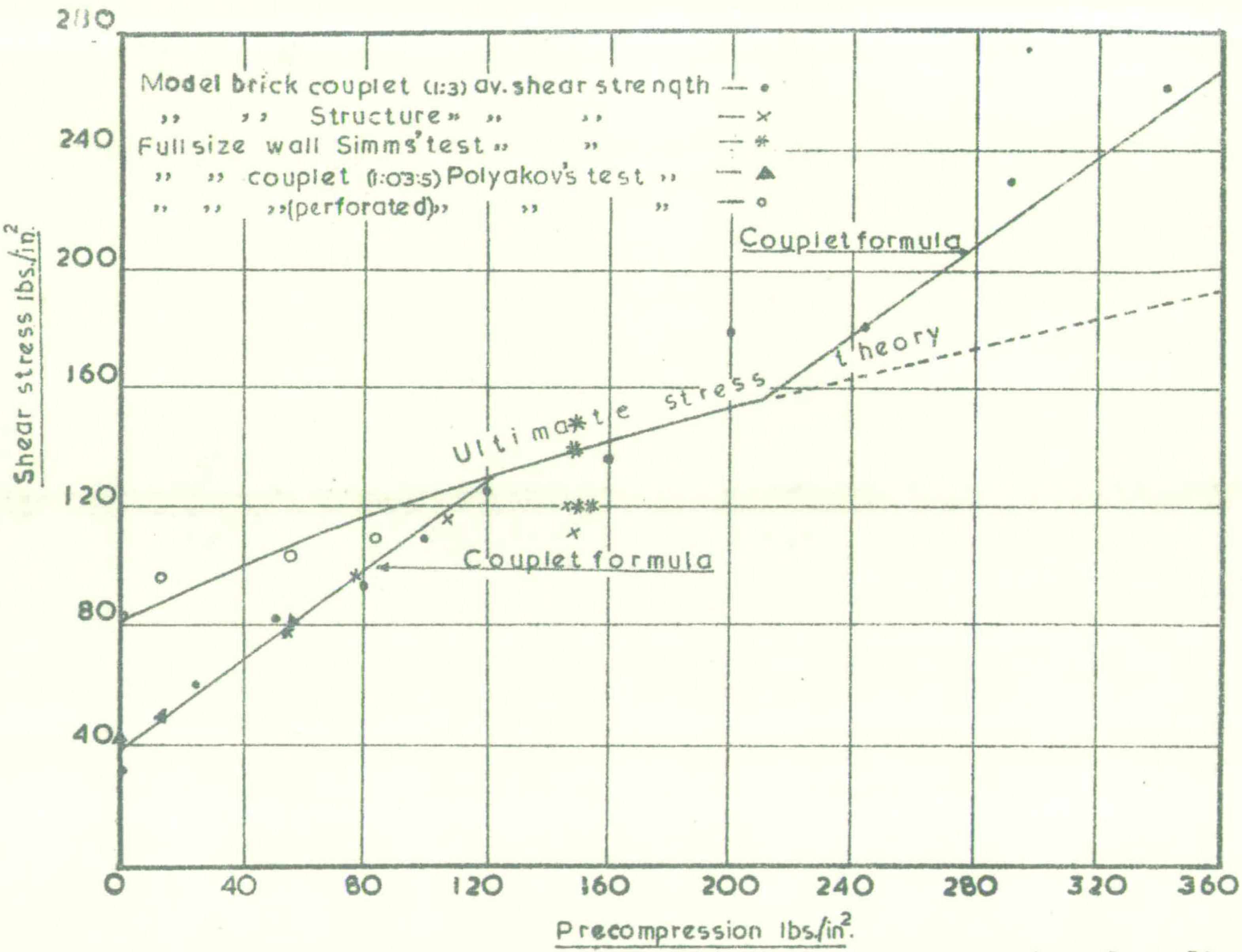


Fig.66. Showing the comparison between theoretical derived curve and experimental results.

noticed[56] when the precompression was increased from 120 to 150 lb/in^2. A similar phenomenon was noticed, while carrying out the couplet tests (Section 5.6.2), and is also apparent from the test results of MURTHY and HENDRY[40]. The shear stress of couplets subjected to 160 lb/in^2 ranged from 105-165 lb/in^2, which overlaps the actual shear stress obtained from the racking test of structures 4 and 5.

In Simm's tests[53], the shear stress in the full-size walls having mortar strengths of 1000 lb/in^2, when subjected to 150 lb/in^2 precompression, ranged from 100 to 150 lb/in^2 with an average of 122 lb/in^2. This of course, is in good agreement with the model test results of 119 lb/in^2 and 104 lb/in^2.

The small increase in shear strength with increase in precompression from 120 lb/in^2 to 150 lb/in^2 is because the limit is reached where failure is governed by the ultimate tensile strength of the brickwork (Fig. 6.6). Also it is clear from Fig. 6.6 that within the limits of precompression σy_1 to σy_2, there is small increase in the shear strength and hence for practical purposes may be assumed constant. In Fig. 6.6 the shear strength based on maximum tensile strength of brickwork has also been drawn for comparison. With perforated bricks, POLYAKOV[46] assumed the straight-line formulae, based on an equation similar to eqn. (12) in Section 6.6 and from it calculated the coefficient of friction as 0.15, which appears to be a very low value for brick/

brick and mortar interface. On examination of his results, it is considered that the failure was governed by the maximum tensile stress theory (Fig. 6.6). At the values of precompression in these tests, the frictional effect was not pronounced and the slope of the curve would give a fictitious value of the coefficient of friction.

From Fig. 6.6, it may be seen that below precompression σy_1, if shear strength of a solid brick wall is calculated by the maximum stress theory, the shear strength will be over-estimated. Above the pre-compressive stress of σy_2 the shear strength will be under-estimated.

It may be noted that if the interface between the brick and mortar was strengthened to make it behave as a homogeneous material and so ensure failure in diagonal tension up to the precompression value of σy_1, this could result in considerable economies in many multi-storey brickwork structures. As observed above, it appears from Polyakov's results[46] that this can be achieved by the use of perforated bricks.

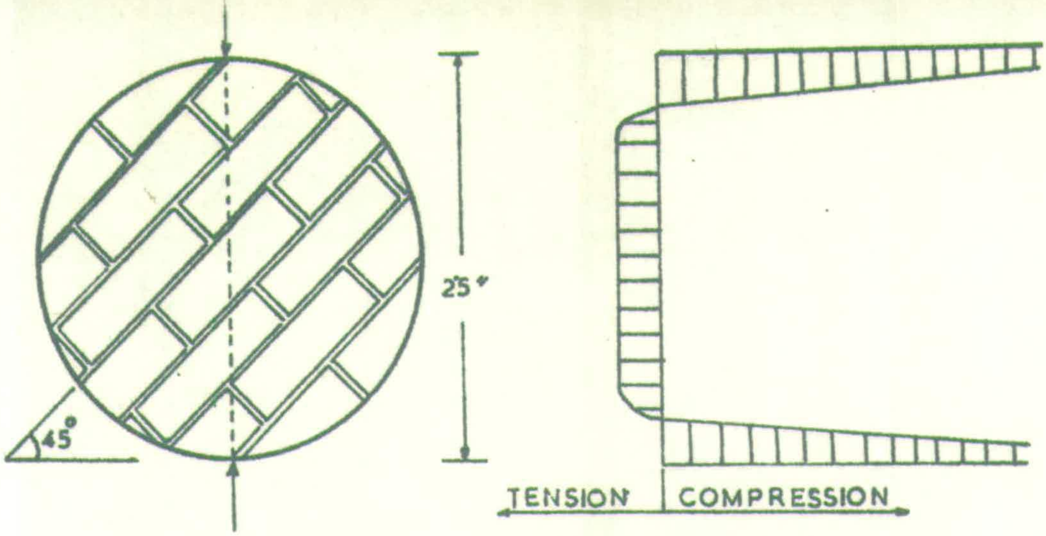
In model structures 3, 4 and 5 the crack passed through some of the bricks. The failure of these structures first started at the interface of the vertical mortar joints of the compression diagonal in the left hand side of panel A (Fig. 6.1 and 6.8 Stage 1). The load was increased slightly and the crack subsequently passed through the bed joints (Fig. 6.8 Stage 2). At this stage the structure was badly damaged and instant failure takes place. At the point of failure, therefore/

therefore, cracks may follow the line of least resistance and develop above and below a particular brick, as suggested in Fig. 6.8, (Stage 3) and final failure may take place through this brick, although the calculated principal tensile strength on the uncracked section may be much below the tensile strength of bricks in diagonal tension.

6.7.1 COMPARISON WITH C.P.111: 1964

The results of the tests are compared with the code of practice as shown in Table 6.1 and Fig. 6.9. The permissible shear stress[10] for walls built with 1:$\frac{1}{4}$:3 or stronger mortar, is 20 lb/in^2. The code allows a proportional increase in shear stress for an increase in vertical stress between 60 and 90 lb/in^2, but no proportional increase or decrease above or below this limit. The code appears to be justified in fixing the upper limit for permissible shear stresses, which will depend very much on the principal tensile stress set up in the brickwork or frictional resistance at the interface of the element. However, beyond this range the shear strength may increase as mentioned previously but the code does not allow any further increase. The shear stress without external precompression (found by BENJAMIN and WILLIAMS[5]) was 16 lb/in^2 and (by POLYAKOV[46]) 45 lb/in^2 in full-size brickwork, and (by MURTHY and HENDRY[40]) 28 lb/in^2 in the model brickwork in 1:3 mortar. POLYAKOV[46] found 0.7 as the coefficient of friction between full-size brick and mortar. Assuming this value of the coefficient of friction to hold good in the Simms' tests[53], the initial bond shear works out to be 17 lb/in^2.

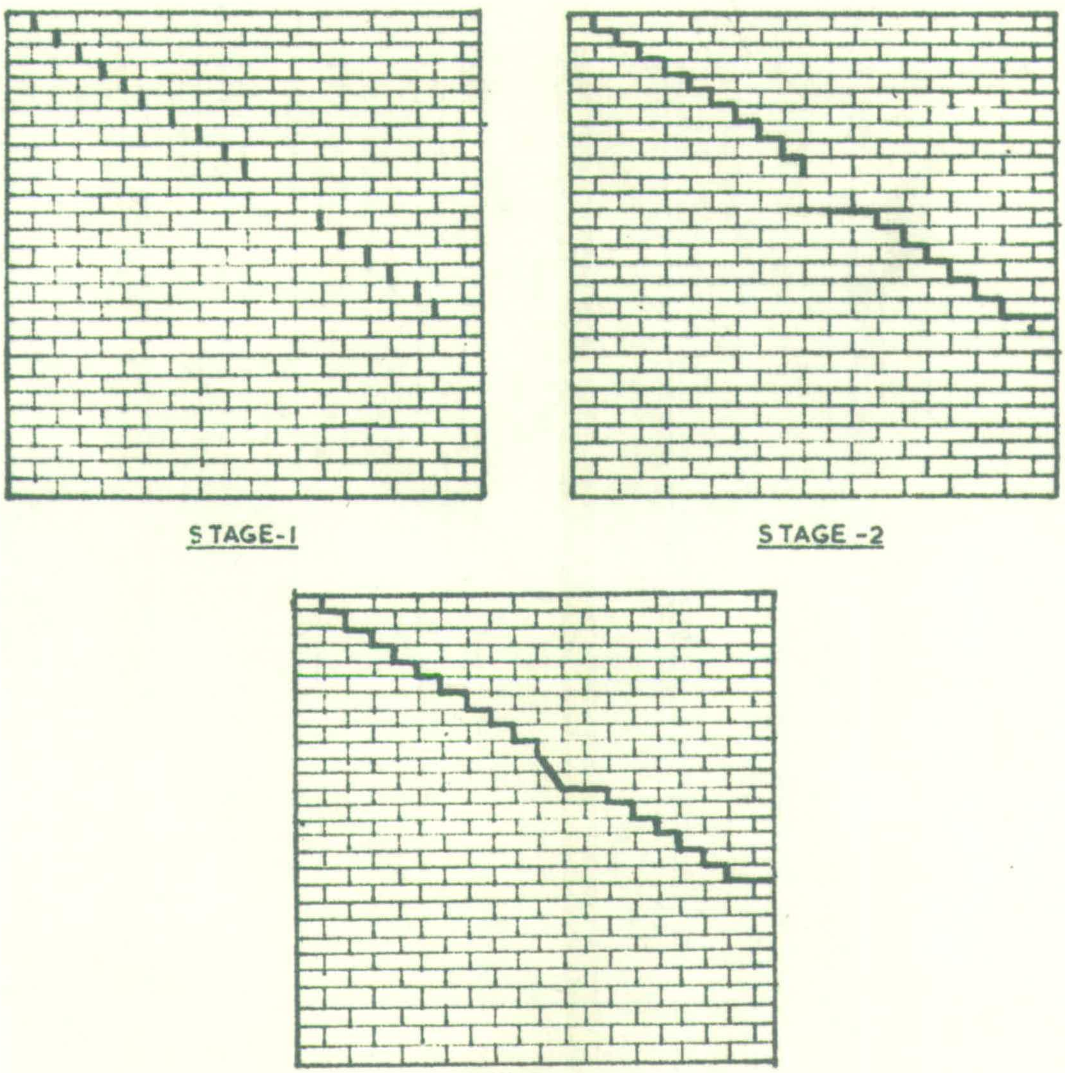
All the results, including those given in this chapter, indicate that the lower limit of shear stress of 20 lb/in^2 recommended by the code[10] without/



Test arrangment for Diagonal tensile strength of brickwork

Stress Distribution

Fig. 6.7.



The Progressive failure of brickwork panel in shear.

Fig. 6.8.

without external precompression is somewhat high, with no safety factor in the case of unperforated wire-cut or single-frog bricks. The shear strength of the brick masonry without precompression depends mainly on the strength of the bond between the brick and mortar. The bond is affected by a great many factors such as consistency of mortar, surface characteristics of the bricks, treatment of bricks before laying, moisture absorption of the bricks and workmanship. It is impracticable to control all these factors on site at all closely.

With this limitation in mind and with a safety factor of 3, based on the tests described in this note, a permissible shear stress of only 15 lb/in^2 without external precompression appears to be more reasonable for solid or single-frog bricks (Fig. 6.9).

Based on the results of the present tests it is also suggested that the permissible shear stress without precompression should be increased by adding one quarter of the vertical stress up to a maximum of 45 lb/in^2 for walls built with solid bricks in 1:3 mortar and subjected to 200 lb/in^2 precompression. Beyond this range, the shear strength may be increased by adding one quarter of the precompression, which will be the maximum allowable compressive stress for the brick in question. This suggestion based on the ultimate shear strength of the model brick structure would give an overall safety factor of 3 at site (Fig. 6.9).

The/

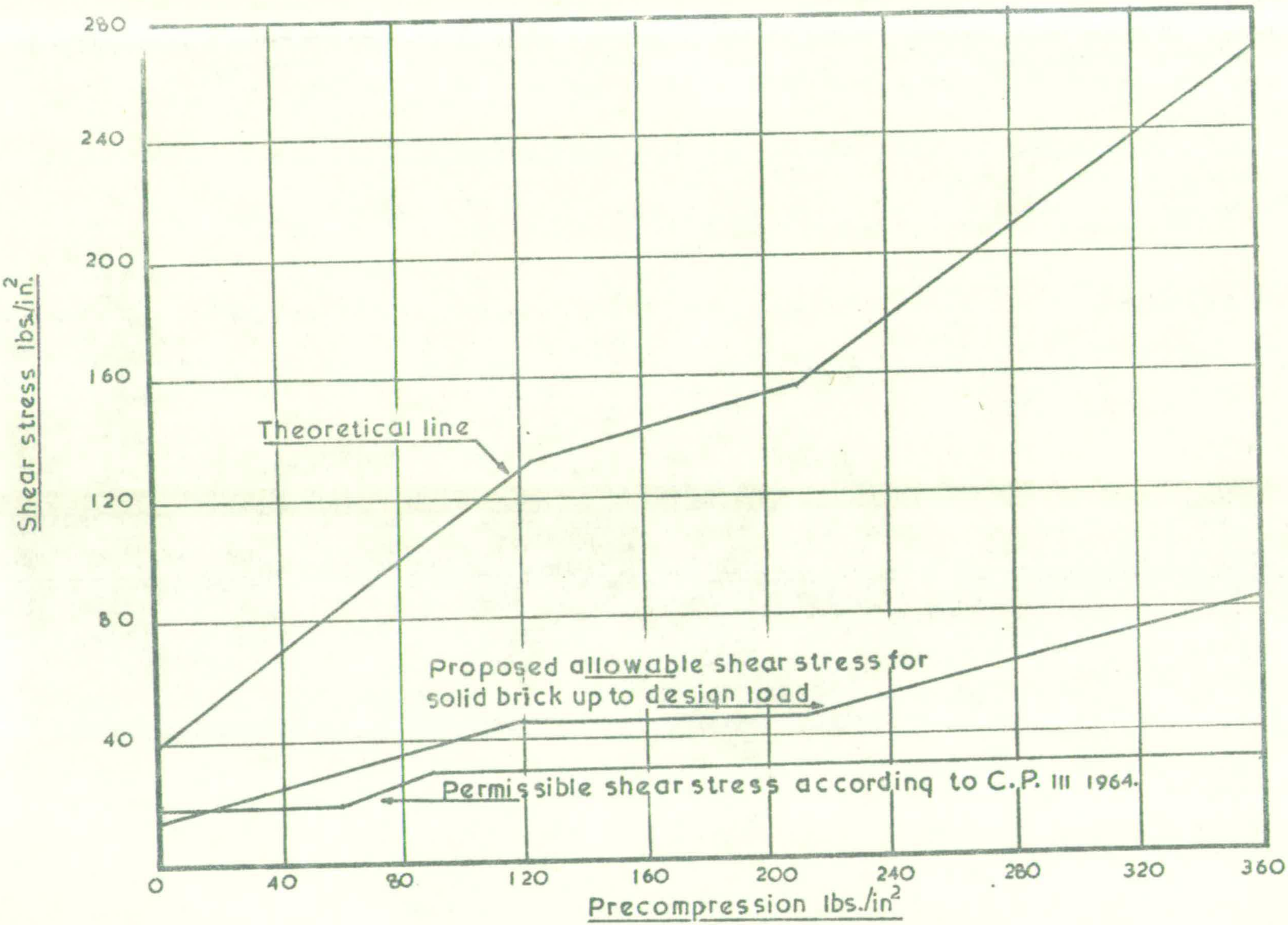


Fig.6.9. - Showing the comparison between the shear strength as per code and experimental results.

The relationship between the racking load, horizontal deflection at slab level and the shearing modulus of the test structures are non-linear. The rigidity and shearing modulus both decrease with an increase of load. The precompression generally increased the rigidity, shearing modulus and the shear strength of the structures, which confirms the test results of MURTHY and HENDRY[40].

In test 3, when the structure was unloaded, cracks were noticed in the slab. It appears that due to repeated loading of the same slab it was damaged during the test and the measured deflection at the slab level was somewhat more than others.

6.8 CONCLUSIONS

1. The theoretical formulae gives a reasonable estimate of the ultimate shear strength of model brickwork structure with and without openings as well as full-size brickwork built of solid bricks. It may be possible to reproduce the ultimate shear strength of full-size brickwork for a given type of brick and mortar by means of model tests (Fig. 6.6).

2. Failure of a storey-height shear wall with openings under a racking load is generally due to breakdown of the bond at the interface, leading to diagonal cracks stepping down through the vertical and horizontal mortar joints or sometimes passing through bed joints only.

3. Over a certain range of vertical compression, the failure of a shear wall occurs in diagonal tension. In this type of failure the crack passes through mortar joints and through some of the bricks.

4./

4. Precompression increases the shear strength of the brickwork up to a certain limit, which may be determined by the compressive strength of the brickwork.

5. The rigidity and shearing modulus decreases with increase of the racking load and the relationship between them is non-linear.

6. The rigidity, shearing modulus and shear strength increase with increase of precompression.

TABLE 6.2

RELATIONSHIP BETWEEN THE RACKING LOAD AND HORIZONTAL DEFLECTIONS AT SLAB LEVEL

Test structure	Racking load W (lb)	Racking load W_1 on 'A' (lb)	Racking load W_2 on 'B' (lb)	Shear stress on 'A' (lb/in^2)	Shear stress on 'B' (lb/in^2)	Horizontal deflection Δ (in. x 10^4)	Rigidity R_2 of 'A' = $\frac{W_1}{\Delta}$ (lb/in x 10^{-4})	Rigidity R_1 of 'B' = $\frac{W_2}{\Delta}$ (lb/in x 10^{-4})	Rigidity of the structure $\frac{W}{\Delta} = (R_1 + R_2)$ (lb/in x 10^{-4})	Shear modulus G (lb/in^2 x 10^{-4})
No.1										
Precompression = 55 lb/in^2	520	380.64	139.36	18.03	13.50	22	6.33	17.30	23.63	22.10
	1300	951.60	348.40	45.07	33.76	131	2.66	7.26	9.92	9.30
	1800	1317.60	482.40	62.41	46.74	228	2.11	5.78	7.89	7.40
	2000	1464.00	536.00	69.34	51.94	332	1.61	4.41	6.02	5.65
	2240 (ultimate)	1639.68	598.00	77.66	57.95	676	0.89	2.42	3.31	3.10
No.2										
Precompression = 78 lb/in^2	500	366.00	134.00	17.33	12.98	14	9.57	26.14	35.71	33.46
	1000	732.00	268.00	34.67	25.97	36	7.44	20.33	27.77	26.09
	1500	1098.00	402.00	52.00	38.95	87	4.62	12.63	17.24	16.15
	2000	1464.00	536.00	69.34	51.94	189	2.84	7.74	10.58	9.91
	2500	1830.00	670.00	86.60	64.90	339	1.98	5.39	7.37	6.90
	2820 (ultimate)	2064.00	756.00	97.77	73.23	652	1.16	3.16	4.32	4.06
No.3										
Precompression = 109 lb/in^2	500	366.50	133.50	17.50	13.07	19	7.03	19.29	26.32	24.97
	1000	733.00	267.00	35.00	26.14	75	3.56	9.77	13.33	12.65
	1500	1099.50	400.50	52.62	39.21	162	2.47	6.79	9.26	8.79
	2000	1466.00	534.00	70.20	52.27	268	1.99	5.47	7.46	7.08
	2500	1832.50	667.50	87.70	65.34	373	1.79	4.91	6.70	6.36
	3000	2199.00	801.00	105.13	78.41	769	1.04	2.86	3.90	3.70
	3280 (ultimate)	2404.24	875.76	115.07	85.73	843	1.04	2.85	3.39	3.69

TABLE 6.1

SHEAR STRENGTH OF ONE-SIXTH-SCALE STOREY-HEIGHT SHEAR-WALL STRUCTURES WITH OPENING AND SUBJECTED TO PRECOMPRESSION

Test No.	Average compressive strength of mortar (lb/in^2)	Normal compressive stress (lb/in^2)	Ultimate racking load as per calculation (lb)	Ultimate shear stress (lb/in^2)	Ultimate shear stress according to couplet formula (lb/in^2)	Max.perm.stress according to C.P. 111: 1964 (lb/in^2)	Safety factor over C.P.111:1964	Max. tensile stress (lb/in^2)	Max. compressive stress (lb/in^2)
1	2172	55	1640	77.66	78.0	20.0	3.88	54.90	109.90
2	1926	78	2064	97.77	96.0	26.0	3.76	61.00	144.00
3	2234	109	2404	115.07	117.0	30.0	3.84	72.50	181.50
4	1881	151	2170	103.84	146.0	30.0	3.45	53.50	204.50
5	1500	147.5	2492	119.30	143.0	30.0	3.97	66.4	214.00

Brick strength = 4332 and 4221 lb/in^2

Co-efficient of friction between bricks and mortar = 0.74.

TABLE 6.2
(continued)

No.4										
Precompression = 151 lb/in^2	500	366.50	133.50	17.50	13.07	11	12.13	33.32	45.45	43.13
	1000	733.00	267.00	35.00	26.14	31	8.61	23.65	32.26	30.61
	1500	1099.50	400.50	52.62	39.21	66	6.07	16.66	22.73	21.56
	2000	1466.00	534.00	70.20	52.27	138	3.87	10.62	14.49	13.75
	2500	1832.50	667.50	87.70	65.34	261	2.56	7.02	9.58	9.08
	2800	2052.40	747.60	98.23	73.19	593	1.26	3.46	4.78	4.48
	2960 (ultimate)	2170.00	790.00	103.84	77.37	-	-	-	-	-
No.5										
Precompression = 147.5 lb/in^2	500	366.50	133.50	17.50	13.07	22	6.07	16.66	22.72	21.49
	1000	733.00	267.00	35.00	26.14	44	6.07	16.66	22.72	21.49
	1500	1099.50	400.50	52.62	39.21	80	4.95	13.57	18.52	17.58
	2000	1466.00	534.00	70.20	52.27	172	3.10	8.52	11.62	11.02
	2500	1832.50	667.50	87.70	65.34	288	2.32	6.36	8.68	8.23
	3000	2199.00	801.00	105.13	78.41	422	1.90	5.20	7.10	6.73
	3400 (ultimate)	2492.00	908.00	119.30	88.88	907	1.00	2.75	3.75	3.56

CHAPTER 7

Investigation of the Behaviour of a Five-Storey Cross-Wall Structure in Brickwork.

7.1 INTRODUCTION:

Recently it has been recognised that masonry can participate in resisting the lateral loads due to wind, earthquakes and atomic blast. For the design of tall structures, the wind load becomes a major criterion and the advantage of shear wall construction and the inherent strength of masonry could not be overlooked. Thus recent years have seen a rapid increase in the use of load-bearing brickwork in form of cross-wall structures. This increase demands a greater knowledge of the behaviour of this type of structure. For the design of such structure a simple method is adopted in which lateral moments are apportioned between the shear walls present in proportion to their flexural rigidities. A more refined method is to take into account interaction between the shear walls and interconnecting floor slabs or beams on the assumption of fully rigid connection between the various elements. The actual behaviour of a brickwork structure is likely to lie between these two extremes and the object of this work was to study behaviour of a typical multi-storey cross-wall structure and to compare the results with those obtained from the existing theories. To this end a 1/6 scale model of a brick cross-wall structure was constructed and loaded at each floor level, in horizontal direction to simulate the windloading as shown in Plate 7.2.

7.2. MATERIALS

7.2.1/

7.2.1 Bricks

One sixth model bricks with an average crushing strength of 4,221; 3,835, and 3485 lbf/in^2 were used in the construction of the walls.

7.2.2 Sand and Cement

The same sand as referred in Section 3.2.2 was used. The rapid hardening cement was used, which conformed to B.S.(12)

7.2.3 Mortar

1:4 Cement sand mortar (1:3 by Vol.) by wt. was used for the construction of all the walls. The result of the crushing of 1" mortar cubes are given in Table 7.1.

Showing the Crushing Strength of Mortar Cubes

TABLE 7.1

Crushing strength lbf/in^2	Range	Mean	Standard Deviation	Co-eff. of variation	Nos.
	1000-2307	1619	272	16%	174

A mortar mix of 1:1 was used for assembling the wall together and joining them to the slab.

7.3 Constructional Details

7.3.1 Wall Panel -

Wall Panels were initially built in jigs as shown in Plate 3.2 (Section 3.3). The walls were assembled as shown in fig.7.1. While assembling/

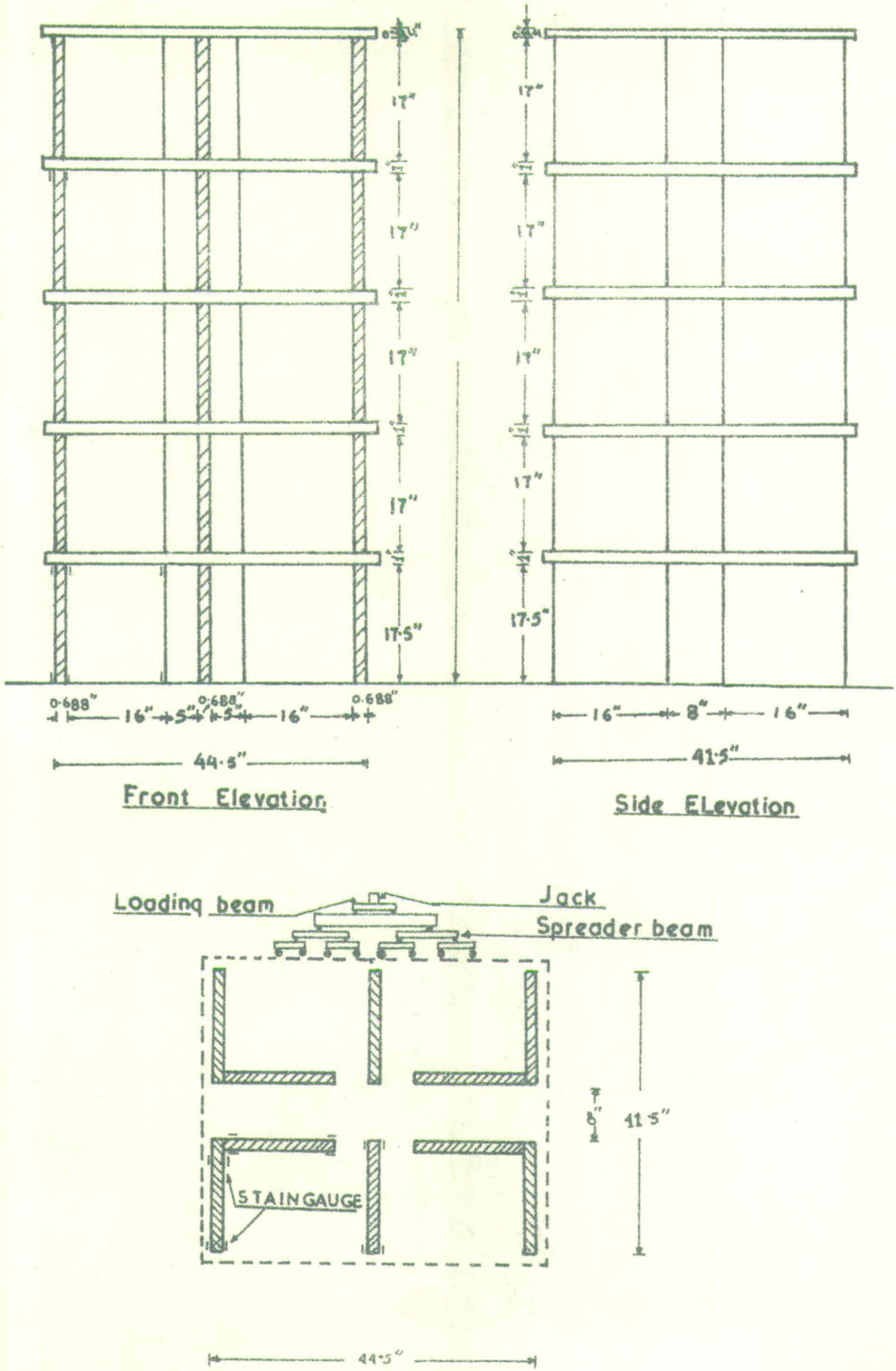


Fig. 7.1 Typical Floor Plan

assembling, care was taken to see that the wall remained plumb and level. The joint between the walls as well as those between the slabs were completely filled with mortar. The bottom of the ground floor walls sit in the receiving channel of the frame, the gaps between walls and channel were filled with mortar. Plate 7.1 shows the arrangement for fixing the wall in position.

7.3.2. R.C. Slab

Each slab was made of 1:1:2 concrete by weight. The maximum size of the aggregate was 3/16". About 1% reinforcement was provided on top and bottom of each slab. The average compressive strength at seven days of 6" dia. x 12"high. cylinders was 5,644 lbf/in^2 and 4" cubes were 7,123 lbs/in^2 respectively.

7.4 Testing Equipment

7.4.1 Loading Frame

The ground floor of the structure was assembled in $1\frac{1}{4}$" steel channel, which was welded to the base frame 8' long by 4' wide made of 4" x 2" channel and 4" x $1\frac{3}{4}$" I section (Plate 7.1). The vertical member of frame consisted of three I sections 8" x 4" size, which was welded on top of I section made of 4" x 2" channel. The base of the vertical frame was connected to the base frame of the movel structure. The frame has the total capacity of 8 tons and was especially designed to test a one-sixth scale five storey-high cross-wall structure by applying horizontal loads to all the storey simultaneously. Provision has been made, for the structure to be loaded from any direction/

7 - Plate 7.1

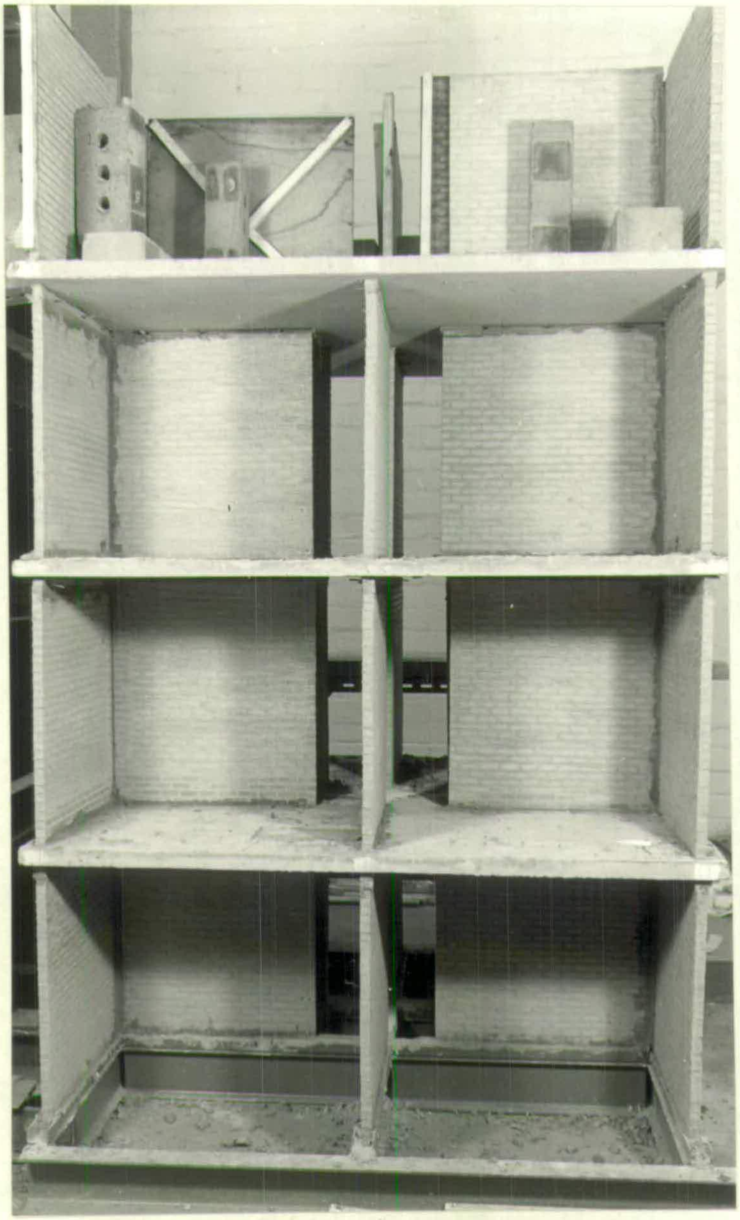


Plate 7.1 - Method of assembling the walls in position

direction.. Plates 7.2 ahow the structure and the loading frame.

7.4.2 Methods of Applying Load

The estimated dead weight stress (50 lbf/in^2) in the lower storey of prototype was simulated by hanging lead billets. The slab weight was also increased by putting lead billets on top of them. The total weight hanging from the walls and placed on the slabs was 2.02 tons. The slab weight was assumed to be distributed evenly on all the walls.

The racking load was applied at each floor level with a 6 tons jack, through a beam 7" x $\frac{3}{4}$" x $\frac{1}{2}$" of high tensile steel supported on rollers 6" apart. The beam details were the same as described in section 6.3.2. The load from each load measuring beam was transmitted to each floor slab through the spreader beam as shown in Fig.7.1. A plywood sheet was put in between the spreads beam and slab to distribute the load evenly.

7.4.3 Load Measuring Apparatus

The racking loads at each storey were measured with 5 numbers of special apparatus of 700 lbs capacity, very similar to one described in Section 6.3.3. The typical calibration curves are shown in Fig.7.2. The calibration was done similarly as described in section 6.3.3.

7.4.4 Details of Test arrangements

Details of test arrangements are shown in plate 7.2 and 7.3. The strains were measured with specially designed vibrating wire strain gauges/

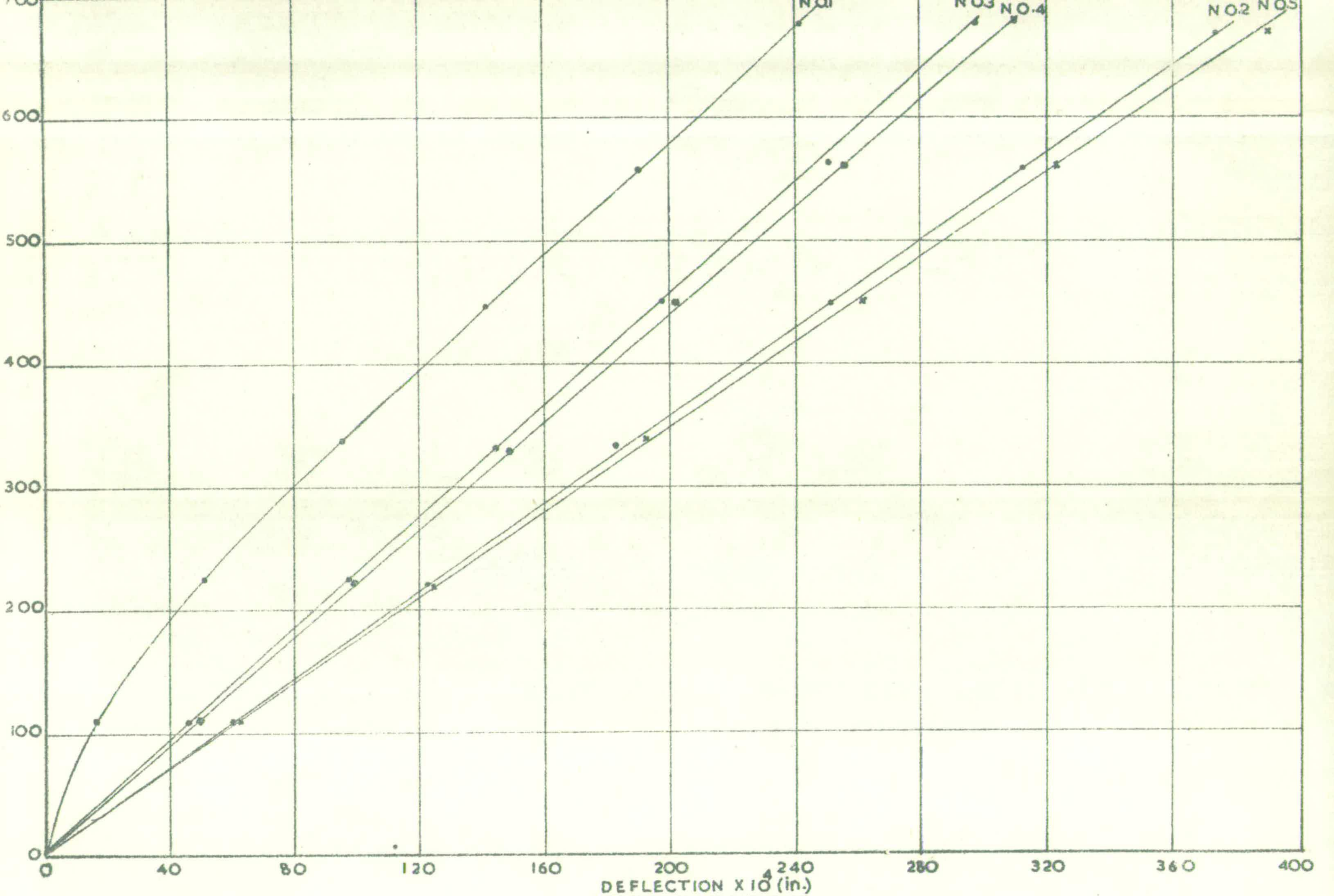


FIG-7.2 - CALIBRATION CURVES FOR LOAD MEASURING BEAMS.

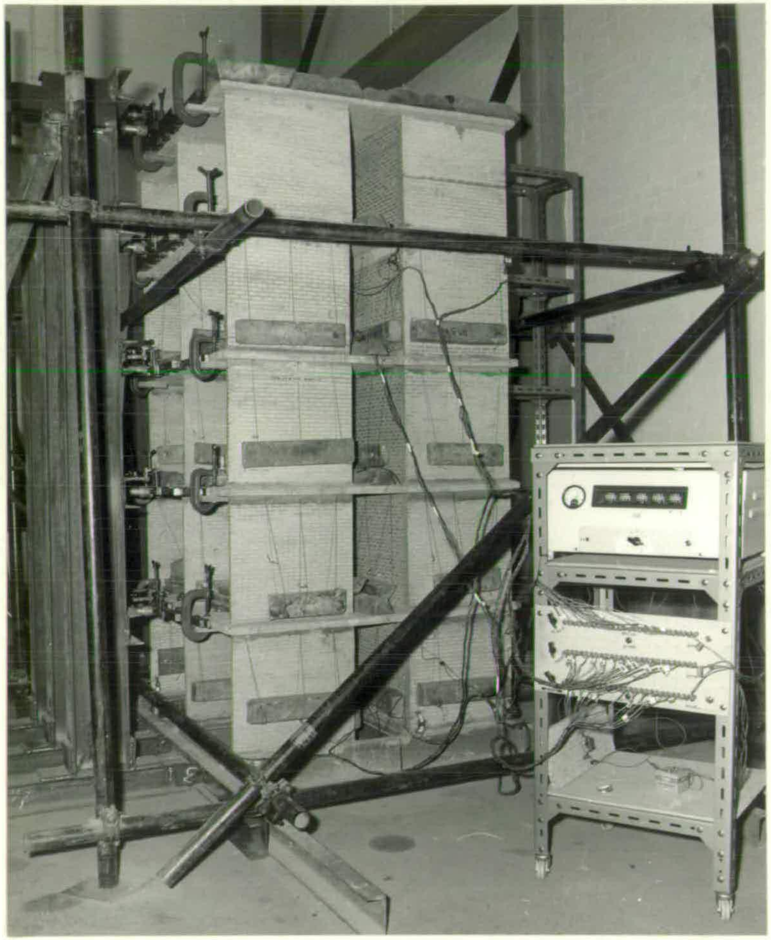


Plate 7.2 - Test arrangement and Loading frame.

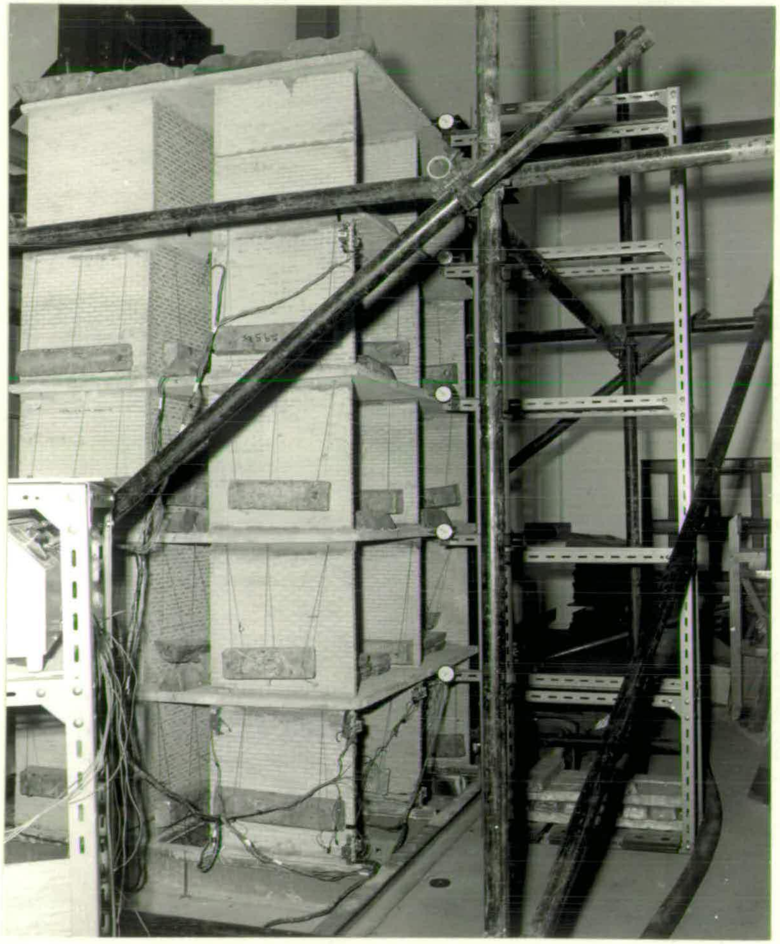


Plate 7.3 - Dial gauges for measuring the deflection.

gauges. The location has been shown in Fig. 7.1 and Plate 7.2 and 7.3. The strains were recorded by Maihak electronic recorder. Each strain gauge was fixed on a standard test piece of steel 3" x 1" x 1" and calibrated in an Avery testing machine against resistance strain gauges fixed on the test sample. The typical calibration curve of some of them are shown in Fig.7.3. While testing dummy gauges were mounted on similar specimen as temperature control.

7.5 Experimental Investigation

Some preliminary experiment in X-X direction (Fig.7.1) was done by loading only at each floor level of the structure with point load and deflections at each floor slabs were measured. The deflection measurements are shown in Table 7.2.(R9).After the preliminary experiment the model was loaded in the X-X direction. The horizontal loads were increased in different stages of the loading and in final stage kept below one third of expected ultimate load. The deflection measurements were taken at each slab level. Two extra dial gauges were mounted at roof level 3 inches from either side of the slab to check if there was any torsion induced in the structure. While testing predetermined loading were applied at each floor level and then all the gauges were read. The strain gauge readings were also noted. Care was taken to see that loads remained fairly constant. The procedure was repeated twice and no appreciable/

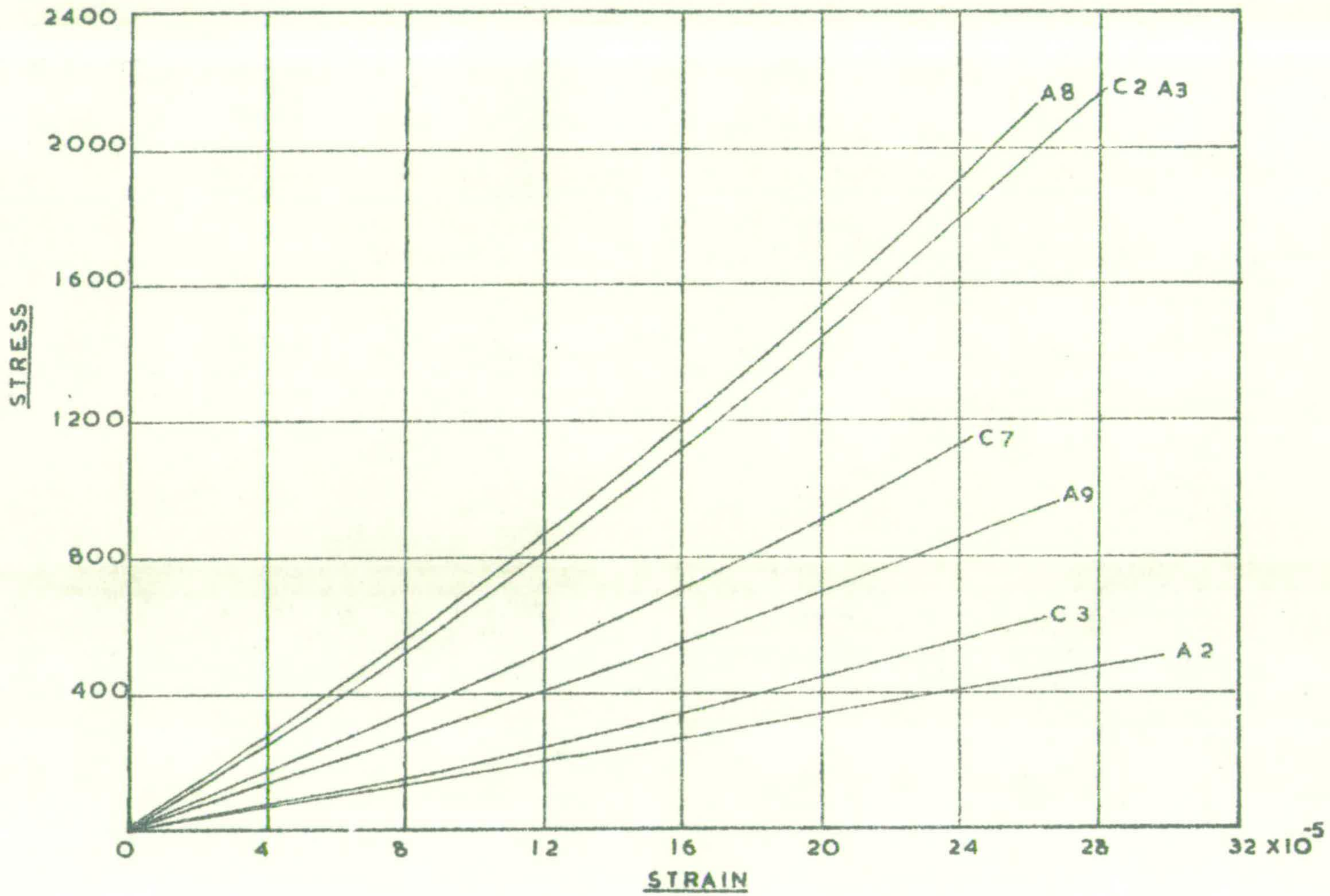


FIG.-7.3 – CALIBRATION CURVES FOR STRAIN GAUGES appreciable difference in deflection was found. No torsion was induced in the structure. The procedure was repeated for the loading in Y-Y direction. Fig. 7.4 and 7.5 show the different loading stages and relationship between the horizontal loads and deflection of the whole structure and at each slab level.

Finally, the model structure was loaded to destruction in Y-Y direction. The first crack at the total load of 1800 lbs similar to the 1st stage of Fig.6.8 (Section 6.7) was noticed in the wall panel of the 1st. storey. This was the shearwall on which strain gauges were earlier mounted (Fig. 7.1). The load was increased slightly and final failure (Plate 7.4 to 7.7) occurred at the ultimate load of 2300 lbs. No damage was noticed in any other storey. Three shear walls (7.4 to 7.7) cracks passed through horizontal and vertical mortar joints. In case of one shearwall (Plate 7.5) failure occurred in the joint between wall and slab. Plate 7.5 and 7.7 shows the failure of cross-wall in loading side. Plate 7.8 shows the distortion of the structure after failure of 1st. storey.

7.6 Design Loading and Factor of Safety

According to the Code of Practice[11] 3, Chapter V, the basic equivalent wind pressure for an equivalent prototype building of 45' height is 17 lbf/ft^2 (Exposure D). In addition to the above wind loading, the shear walls are supposed to resist lateral force equal to 2½% of the total load carried by the Crosswalls[10], which is equivalent to 1.6 lbf/ft^2. So the structure must be designed to resist the lateral/

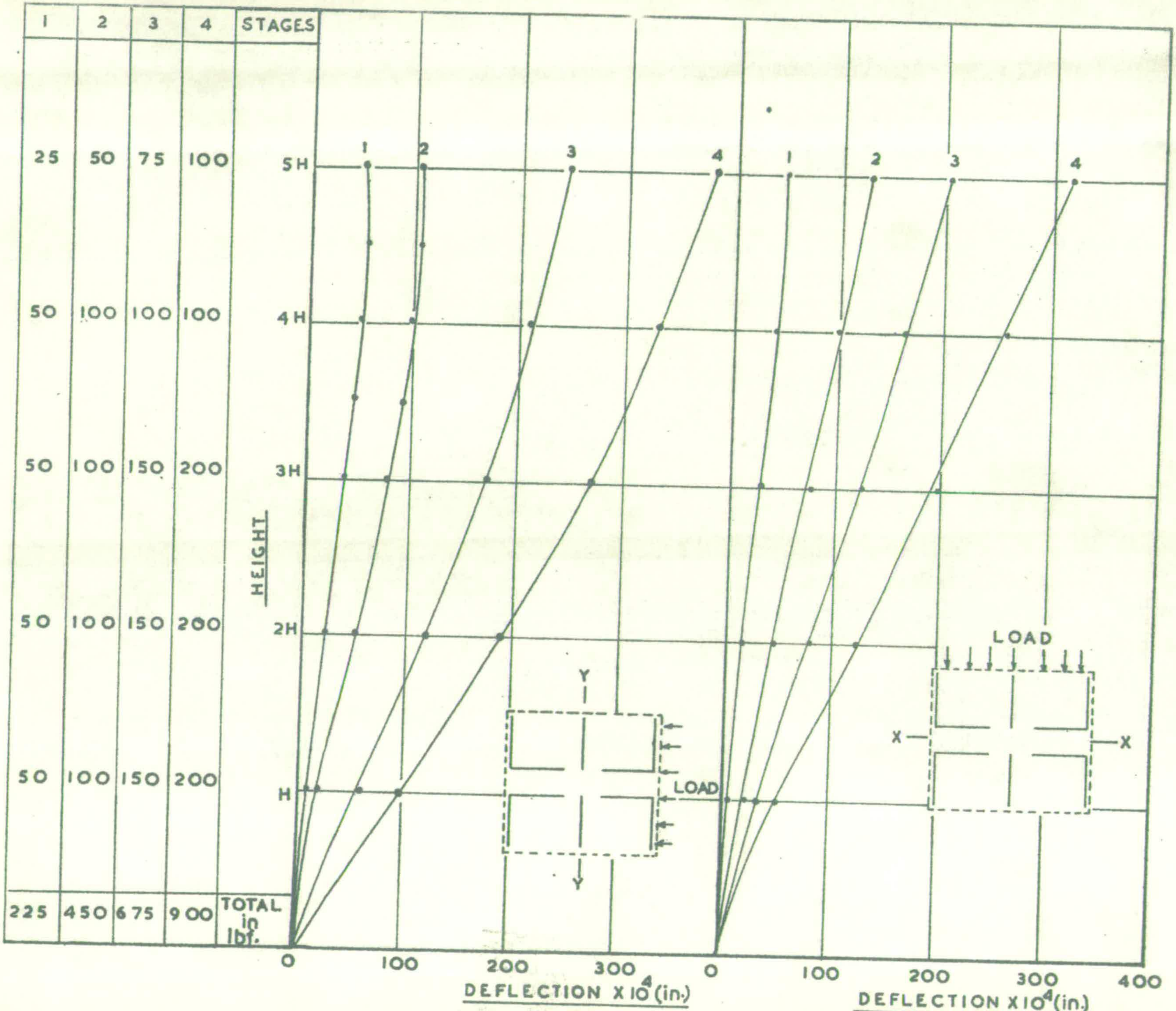


FIG.NO-7.4 DEFLECTION OF THE STRUCTURE AT VARIOUS STAGES OF LOADING

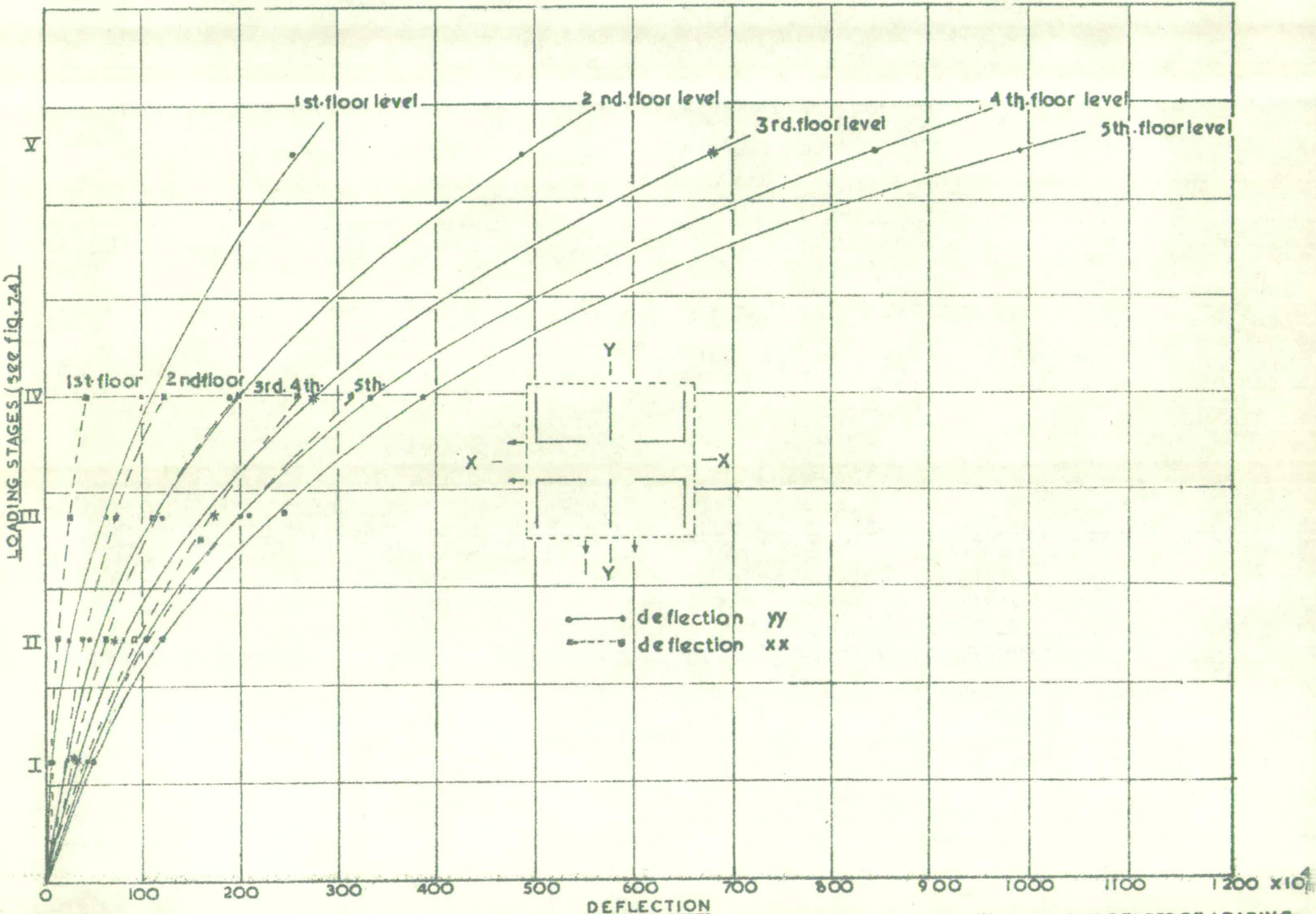


FIG:7.5 RELATIONSHIP BETWEEN DEFLECTION AT VARIOUS FLOOR LEVEL AND HORIZONTAL LOADS IN DIFFERENT STAGES OF LOADING.

7 - Plate 7.4 - 7.5

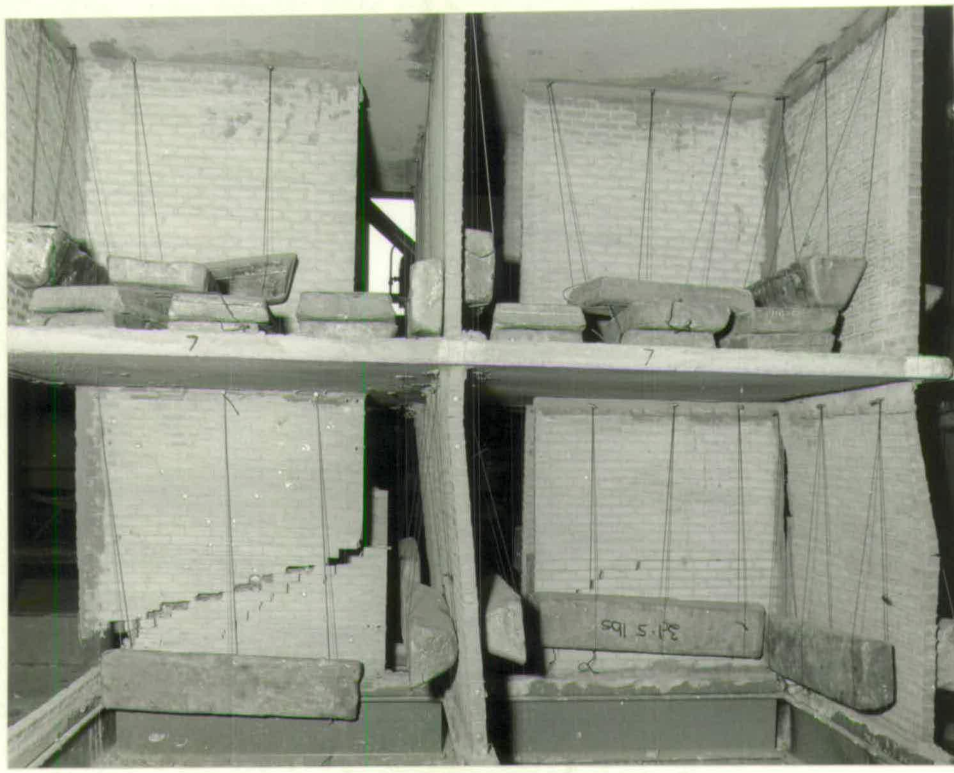


Plate 7.4 - Showing the failure of Shear walls (2 Nos.) in first storey.

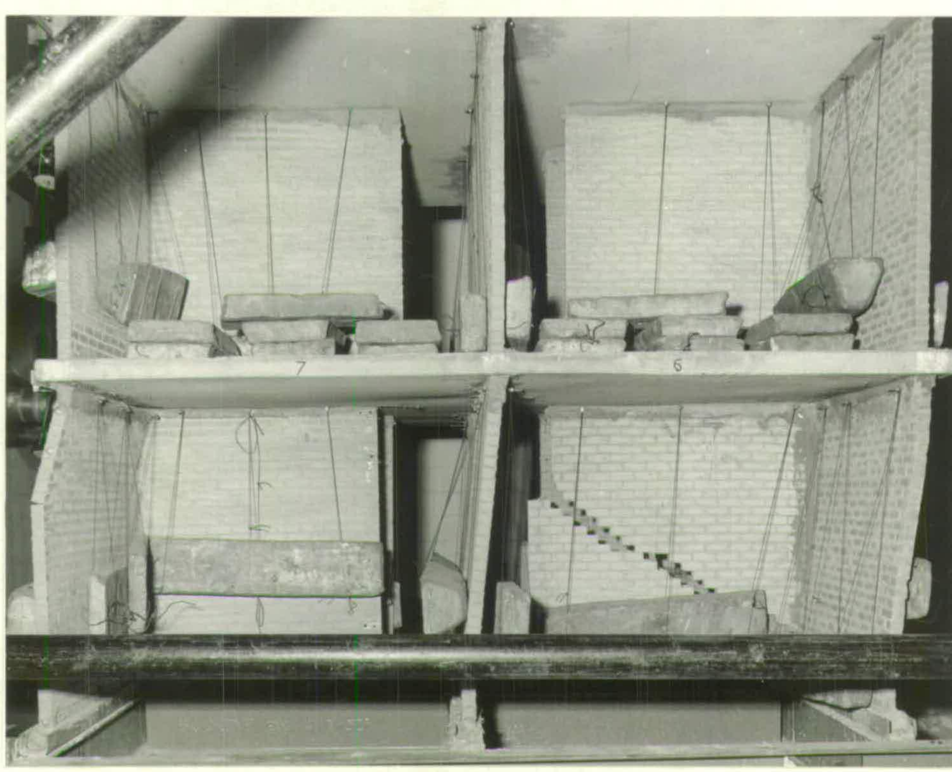


Plate 7.5 - Showing the failure of shear walls (another two) in first storey.

7 - Plate 7.6 - 7.7

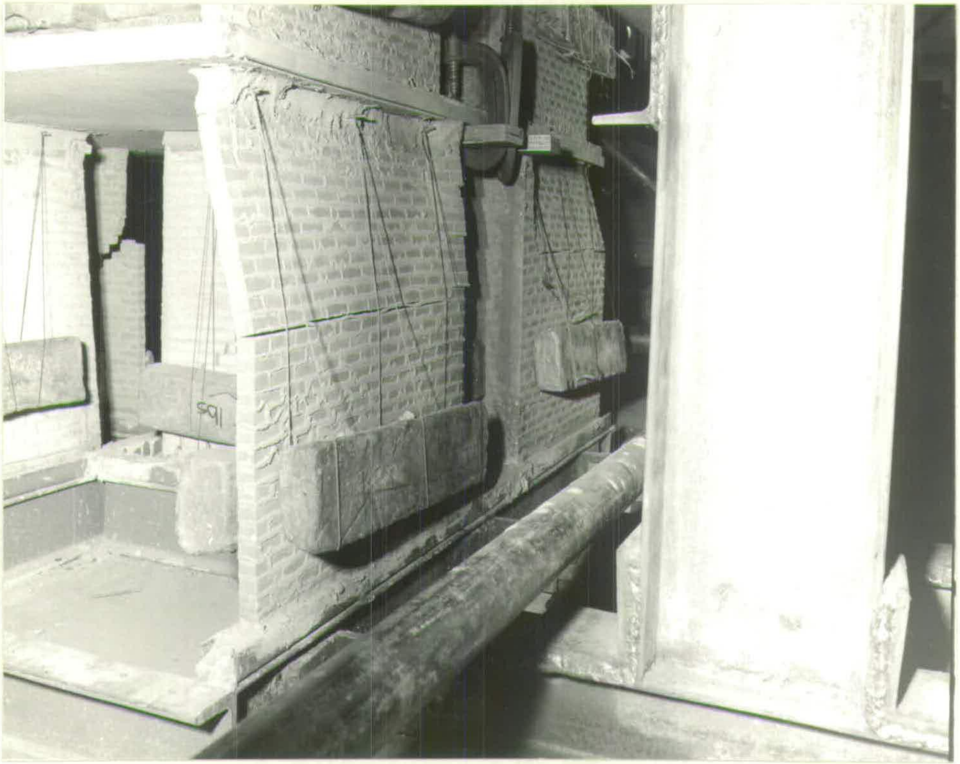


Plate 7.6 - Failure of Cross-wall towards loading frame.

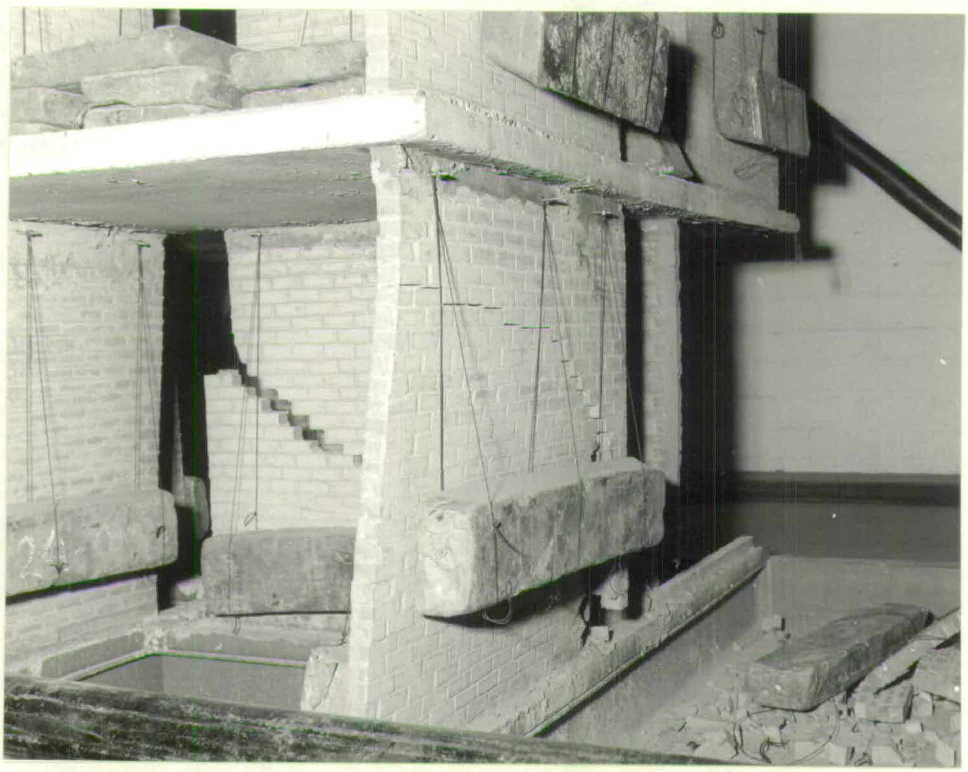


Plate 7.7 - Typical failure of cross-walls in first storey.

7 - Plate 7.8

Plate 7.8 - Distorted shape of the cross-wall structure after failure.

lateral load of 18.6 lbf/ft^2. The ultimate load of 2300 lbs. will be equivalent to uniformly distributed loading of 88.8 lbf/ft^2. This has been obtained by dividing the total load by the surface area exposed to the load. The factor of safety over the design load works out to be 4.6 for a 75 m.p.h. wind.

7.7 Experimental Investigation of Elastic modulii

For any comparison of the experimental results with the available analytical method, the value of elastic modulii of brickwork and concrete were essential. For finding the value of modulus of elasticity of concrete 6" x 12" cylinders were made and loaded in compression. The strains were measured at four places diametrically opposite to each other. The stress strain curve was plotted and tangent to the curve was drawn as shown in Fig.7.6. The average initial tangent modulus of elasticity was found to be 3×10^6. From Fig.7.6 it could be seen that the strain measured in the beginning was somewhat high, this may be due to the closing of microcracks or due to the reduction of voids.

For determining the modulus of elasticity of brickwork a 15.5" long T-beam was made and loaded as shown in Fig.7.7. The deflection was measured with the help of dial gauges and from the load deflection curve (Fig. 7.7) the value of E was found as $.98 \times 10^6$.

7.8 Analysis of the Structure: Three approximate methods were used for the comparison of deflection of the structure:

1. Individual cantilever[59]
2. Continuum[18,24,59,49]
3. Wide Column frame[25]

7.8.1/

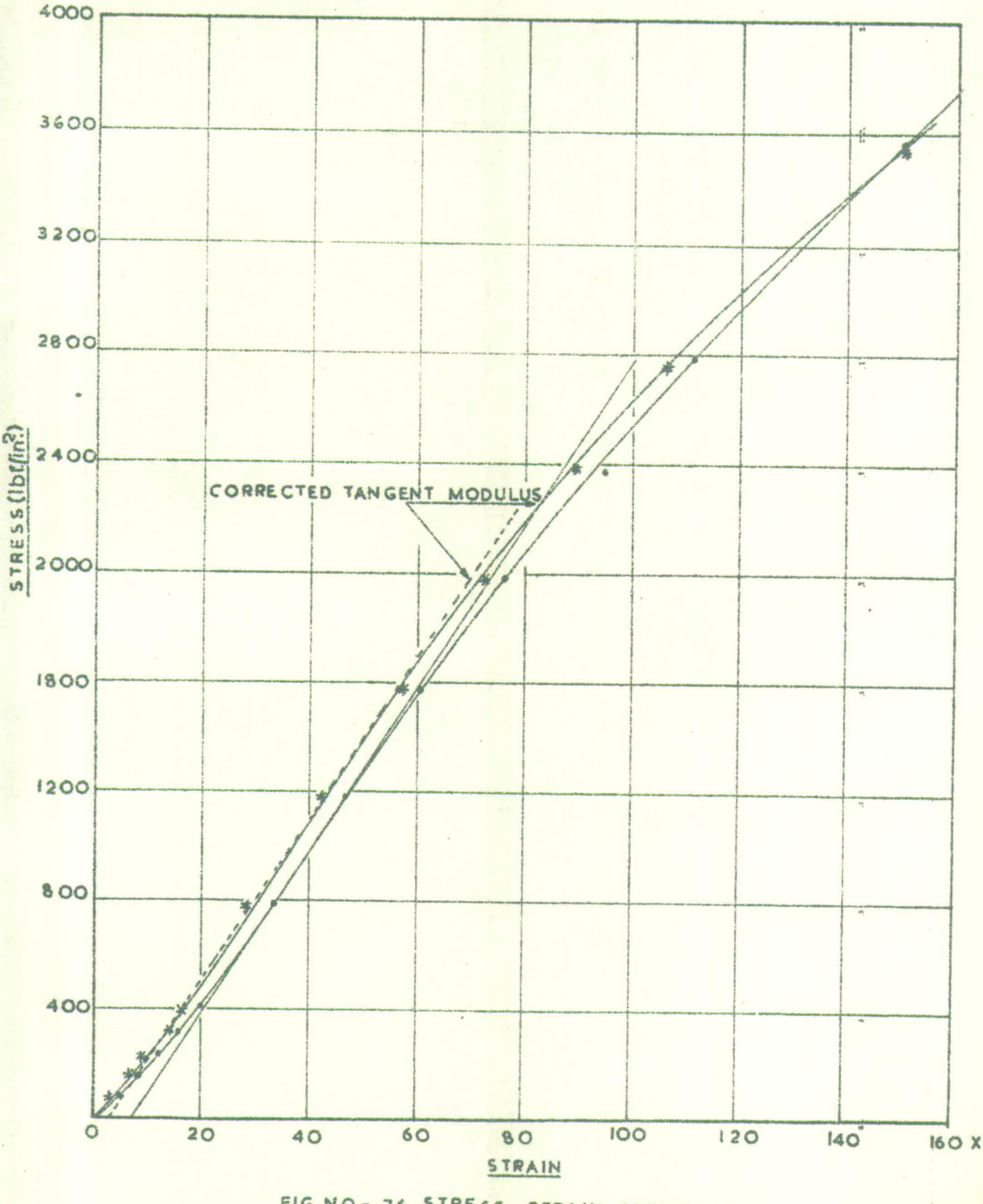


FIG. NO - 7.6 STRESS STRAIN CURVE FOR CONCRETE

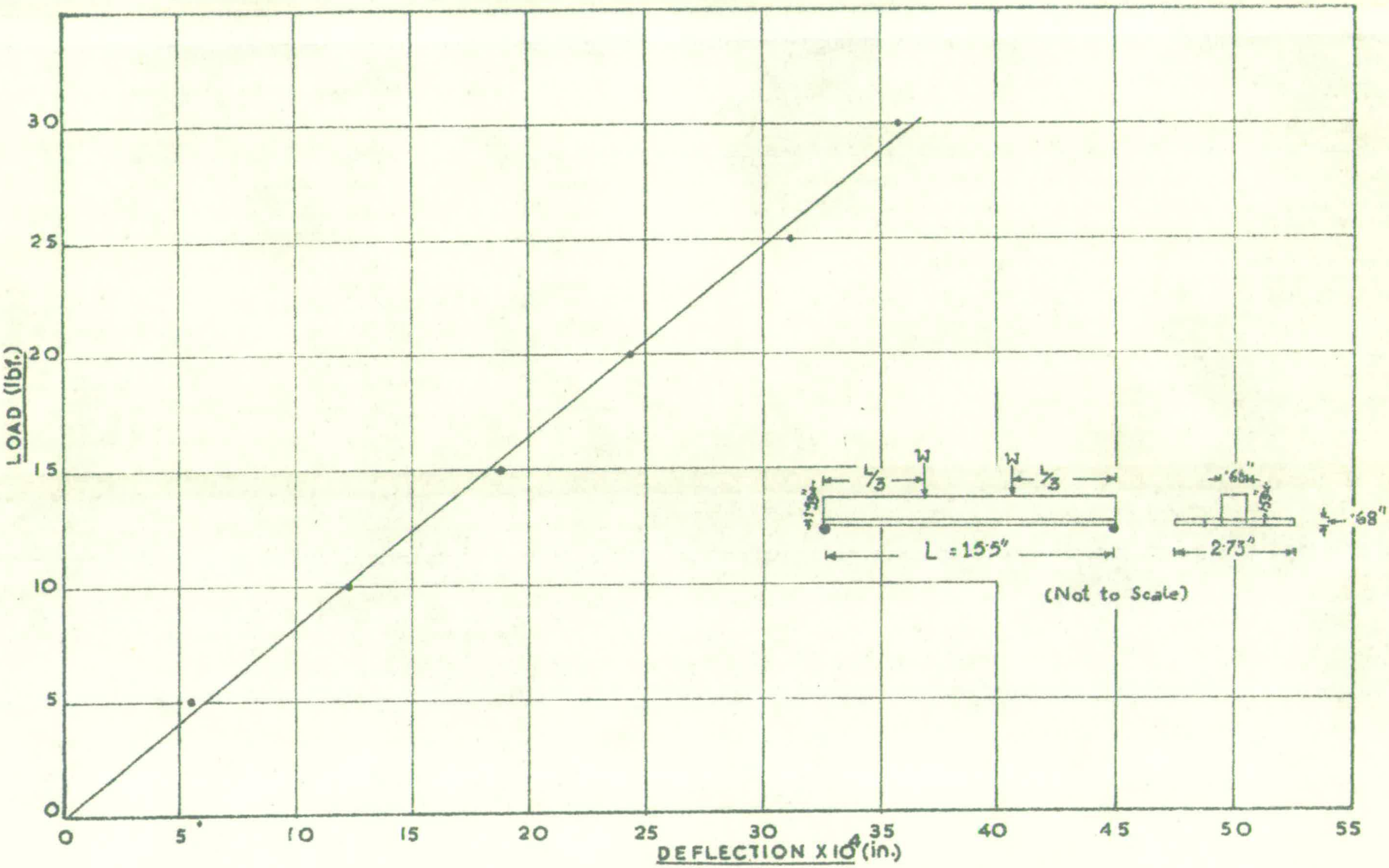


FIG.-7.7 LOAD DEFLECTION CURVE FOR BRICKWORK BEAM

7.8.1 Individual Cantilever:

The assumptions made in the section 6.4.2 holds good in this case also. As the moments of inertia of the slab are negligible compared to the walls, the slab is assumed to act as a strut pin connected to the wall. Each unit is assumed to bend about its own centroid and equivalent moment of inertia of the structure is:-

$$I_e(xx) = I_{1x} + I_{2x} + I_{3x} \quad \ldots\ldots\ldots \quad + I_{nx} \quad \ldots\ldots\ldots \quad (i)$$

and external moment M_o is:

$$M_{ox} = M_1x + M_{2x} + M_{3x} \quad \ldots\ldots\ldots \quad + M_{nx} \quad \ldots\ldots\ldots \quad (ii)$$

Similarly,

$$I_e(YY) = I_{1y} + I_{2y} + I_{3y} \quad \ldots\ldots\ldots \quad + I_{ny} \quad \ldots\ldots\ldots \quad (iii)$$

$$M_{oy} = M_{1y} + M_{2y} \quad \ldots\ldots\ldots\ldots\ldots \quad + M_{ny} \quad \ldots\ldots\ldots \quad (iv)$$

Hence the deflection of the cantilever for distributed load w will be

$$Y_{xx} = \frac{w}{EI_{exx}} \left(\frac{x^4}{24} - \frac{L^3x}{6} + \frac{L^4}{8} \right) - (v)$$

or

$$Y_{yy} = \frac{w}{EI_{eyy}} \left(\frac{x^4}{24} - \frac{L^3x}{6} + \frac{L^4}{8} \right) (vi)$$

The shear deflection has been neglected as the H:L ratio wa 5:1. The I_{exx} and I_{eyy} were calculated from the simplified structure shown in Figs. 7.11, 7.12 and 7.13. The deflections obtained were shown in Fig. 7.11, 7.12 and 7.13.

7.8.2 Continuum:

Following Soane[59] the three dimensional structure has been replaced by a hypothetical structure with appropriate areas and centroids as shown in Fig. 7.8. The slabs have been replaced by an equivalent medium and point/

point of inflection has been assumed in the centre of the connecting medium. Further the moments and shear were assumed to distribute in proportion to rigidity.

With these assumptions the redundant shear force in the lamina will be expressed according to Coull and Chowdhary[18]:-

$$\frac{d^2T}{dx^2} - \alpha^2 T = -\beta x^2 \qquad - (vii)$$

$$\text{where } T = \int_0^x q\, dx \qquad - (viii)$$

$$\alpha^2 = \frac{12\, I}{hb^3}p\left(\frac{l^2}{I} + \frac{A}{A_1 A_2}\right) \qquad - (ix)$$

$$\beta = \frac{1}{2}\, wl \left(\frac{12\, I}{hb^3}p\right)\frac{1}{I} \qquad - (x)$$

$$I_{exx} = I_{1x} + I_{2x} \qquad - (xi)$$

$$A = A_{1x} + A_{2x} \qquad - (xii)$$

By putting appropriate boundry condition, the value of T becomes:-

$$T = \frac{2\beta}{\alpha^4}\left\{\frac{1 + \text{Sinh}\,\alpha H - H\alpha}{\text{Cos h}\,\alpha H} . \text{Sin}\,\alpha H - \text{Cos h}\,\alpha x + \tfrac{1}{2}\alpha^2 x^2\right\} \qquad - (xiii)$$

Once the redundancy of system is found out the bending moments in the walls 1 2 2 are given by:-

$$M_{1x} = \left(\tfrac{1}{2}\, wx^2 - Tl\right)\frac{I_{1xx}}{I_{exx}} \qquad - (xiv)$$

$$M_{2x} = \tfrac{1}{2}\left(Wx^2 - Tl\right)\frac{I_{2xx}}{I_{exx}} \qquad - (xv)$$

and the deflection will be:-

$$EI_{exx}\frac{d^2y}{dx^2} = \tfrac{1}{2}\, wx^2 - Tl \qquad - (xvi)$$

By/

By solving this with appropriate boundry condition given by:-

$$y_{xx} = \frac{1}{2}\ \frac{wH^4}{EI_{xx}} \left\{ \left[\frac{1}{4} - 1/3\ \frac{x}{H} + \frac{1}{12}\left(\frac{x}{H}\right)^2\right]\left[1 - \frac{1}{\mu}\right] - \frac{2}{\mu}\ \left[\frac{\frac{x}{H}^2 - 1}{2(\alpha H)^2}\right.\right.$$

$$\left.\left. + \frac{\alpha H\ (\mathrm{Sin}\,h\alpha H - \mathrm{Sin}\,h\alpha x) - \cos h\alpha(H - x) + 1}{(\alpha H)^4\ \ \mathrm{Cos}\ h\alpha H}\right]\right\} - \text{(xvii)}$$

$$\text{where}\ \mu = 1 + \frac{A}{A_{1x}\ A_{2x}}\frac{I_{exx}}{l^2} \qquad \text{(xviii)}$$

Similarly, the deflection on the other direction can be calculated. The deflections obtained by this method are also shown in Fig. 7.11, 7.12 and 7.13.

7.8.3 Wide Column Analogy

In this case also the structure has been replaced by a hypothetical structure like continuum approach as shown in fig. 7.9, No axial shortening is assumed to occur in the beam, the load acts on the slab level and strain energy due to the axial and shear forces may be neglected. For horizontal loading, due to symmetry of structure in xx and yy direction point of inflection occurs at the centre of the slab, which reduces the degree of indeterminacy. Influence co-efficient method was used to find out the shear force in the connecting members, the derivation of the theory is given in any standard book elsewhere[25].

The values in YY and XX directions may be written in equation form:-

$$\begin{aligned}
99.87y_1 + 49y_2 + 49y_3\ \ldots\ldots\ldots + 49y_5 &= 12774.23 \\
49.0\,y_1 + 148.87y_2 + 98y_3\ \ldots + 98y_5 &= 20563.39 \\
49.0y_1 + 98.0y_2 + 246.87y_3 + 147y_4 + 147y_5 &= 24613.75 \\
49.0y_1 + 98.0y_2 + 147.0y_3 + 393.87y_4 + 196y_5 &= 26171.58 \\
49.0y_1 + 98.0y_2 + 147.0y_3 + 196y_4 + 589.87y_5 &= 26483.15
\end{aligned} \qquad \text{xix}$$

$$31.24\ x_1 + 9.91\ x_2 \ldots\ldots\ldots\ldots + 9.91\ x_5 = 4761.20$$
$$9.91\ x_2 + 41.15\ x_2 + 19.82\ x_3 \ldots\ldots + 19.82\ x_5 = 7664.37$$
$$9.91\ x_1 + 19.82x_2 + 60.97\ x_3 + \ldots\ldots + 29.73\ x_5 = 9174.02 \qquad xx$$
$$9.91\ x_1 + 19.82x_2 + 29.73x_3 + 90.70x_4 + 39.64x_5 = 9754.65$$
$$9.91\ x_1 + 19.82x_2 + 29.73x_3 + 39.64x_4 + 130.34x_5 = 9870.78$$

Now from these, the values of unknowns were found out and the deflections were obtained from the following equations:

$$xx = \int_s \frac{m_{xx}}{EI_{exx}} \left\{ mo + m_1 x_1 + \ldots\ldots\ldots + m_n x_n \right\} d_s \qquad \ldots.(xxi)$$

$$Y_{yy} = \int_s \frac{m_{yy}}{EI_{eyy}} \left\{ mo + m_1 y_1 + \ldots\ldots\ldots m_{y_n} x_{y_n} \right\} d_s \qquad \ldots.(xxii)$$

The deflections obtained are shown in Fig.7.11, and 7.12.

7.9 Analysis of Deflection Curve

Having analysed the idealised structure by available methods, it appeared that these methods could not possibly be used for the design of brick cross-wall structure. Neither the values, nor even the basic form of the curve was obtained. Because of the construction, the brick cross-wall structure could be visualised as discontinous stack of stories put one on top of the other and held together by precompression. The wind load from each storey is transferred due to friction and resisted by friction and racking resistance of wall. The relative displacement of each storey is also prevented by friction at the joint between slab and wall. All other assumptions made in Chapter 6, Section 6.2 hold good. The deflection of each storey will be:-

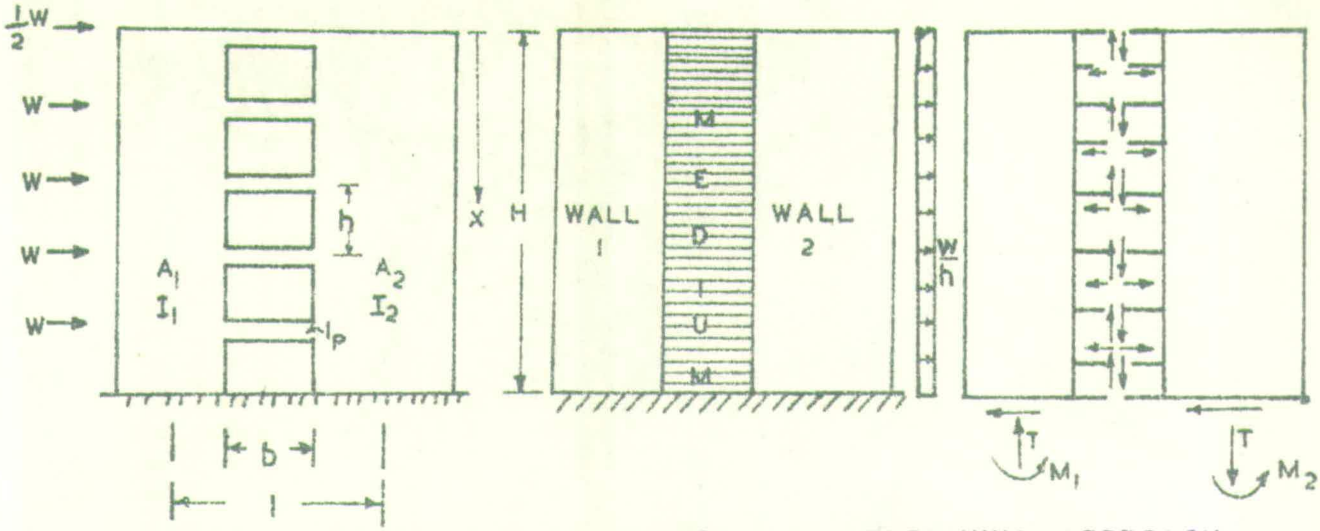


FIG. 7.8 IDEALISED STRUCTURE IN CONTINUUM APPROACH.

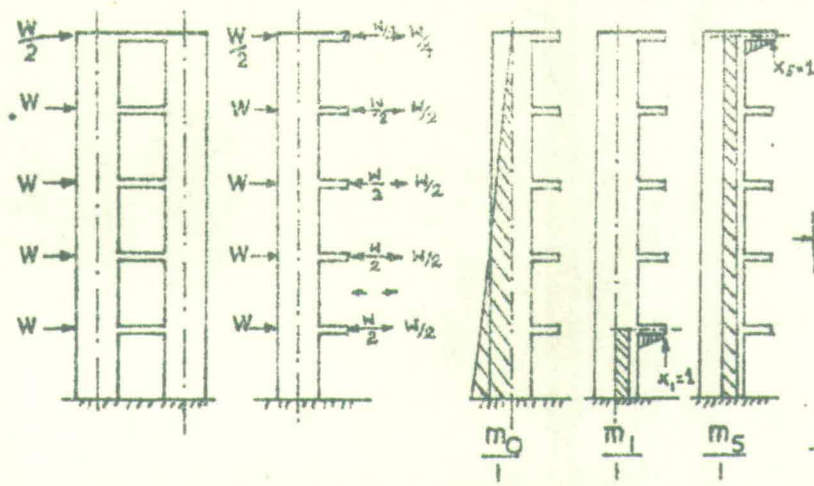
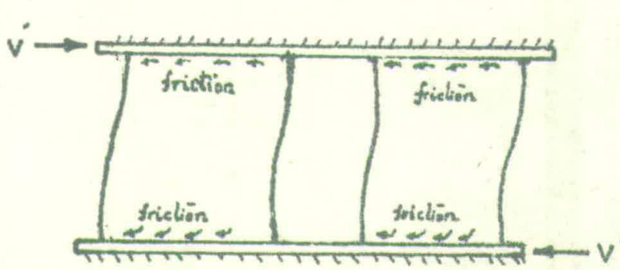


FIG. 7.9 IDEALISED STUCTURE IN WIDE-COLUMN APPROACH



DISTORTION OF A STORY

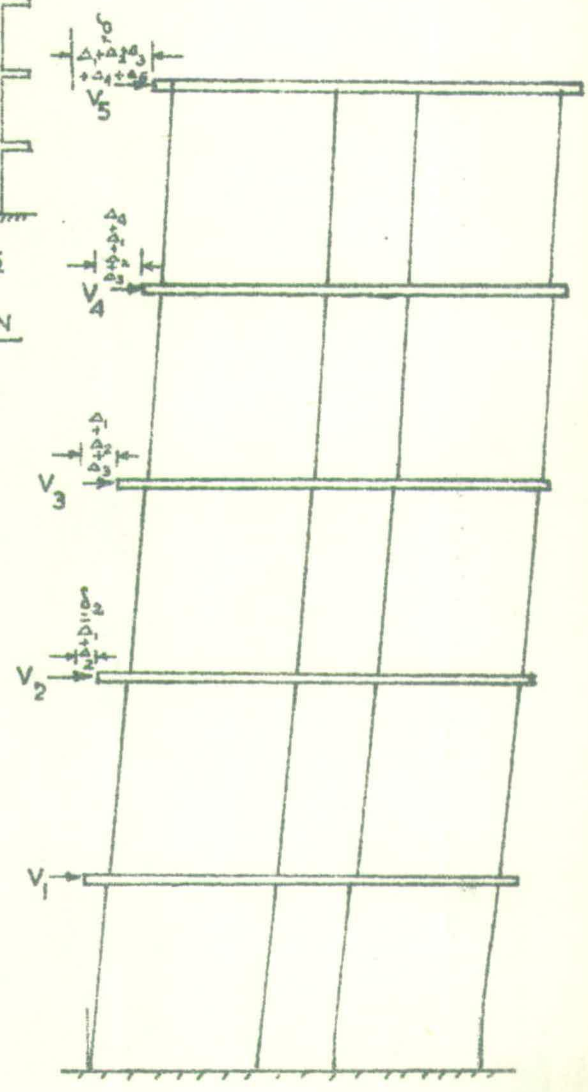


FIG. 7.10 DEFLECTION IN BRICK MULTY-STORY BUILDING

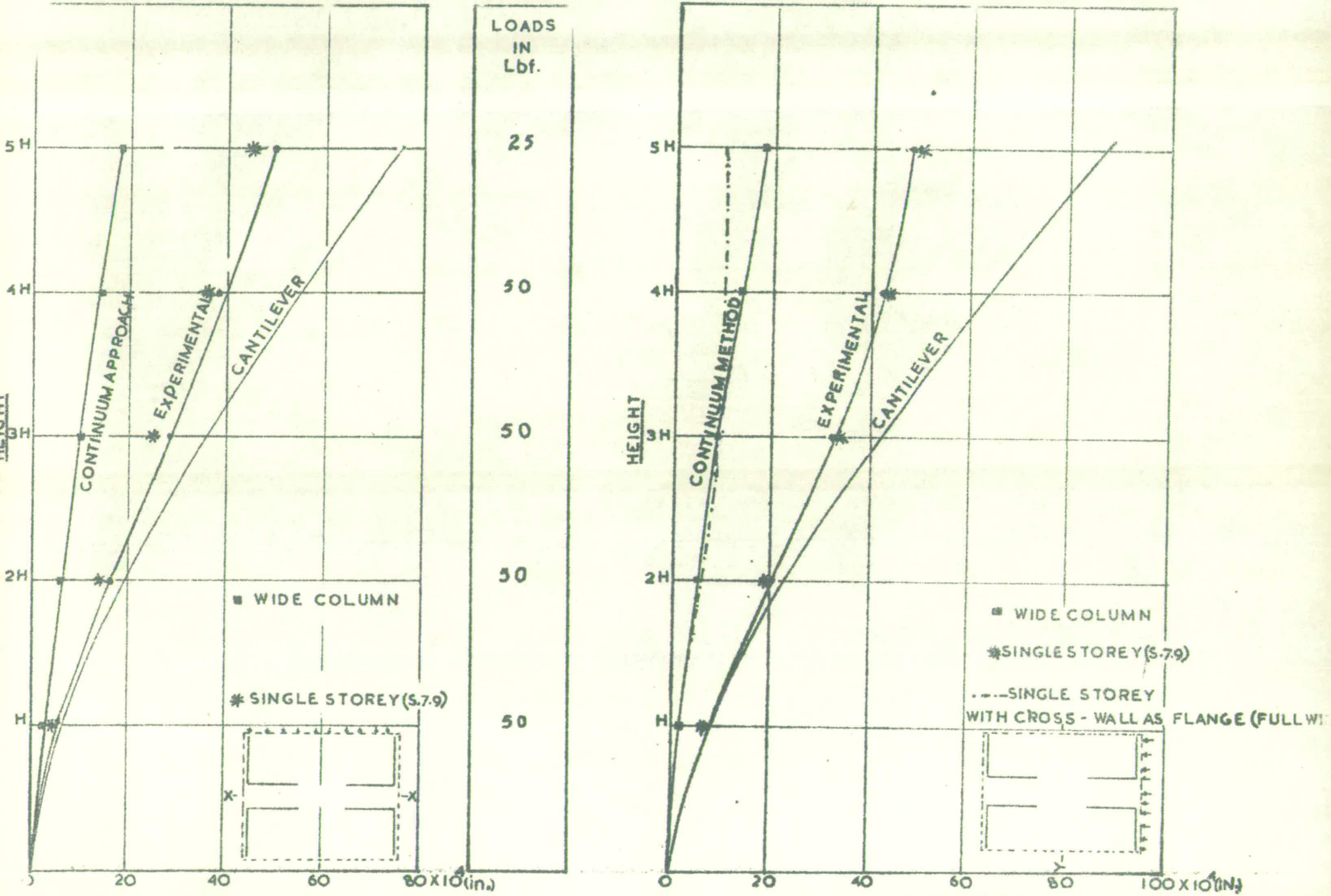


FIG. 7.11 DEFLECTION

FIG. 7.12 DEFLECTION

COMPARISION OF DEFLECTION OF THE STRUCTURE OBTAINED FROM DIFFERENT ANALYTICAL METHODS AND EXPERIMENTAL RESULTS

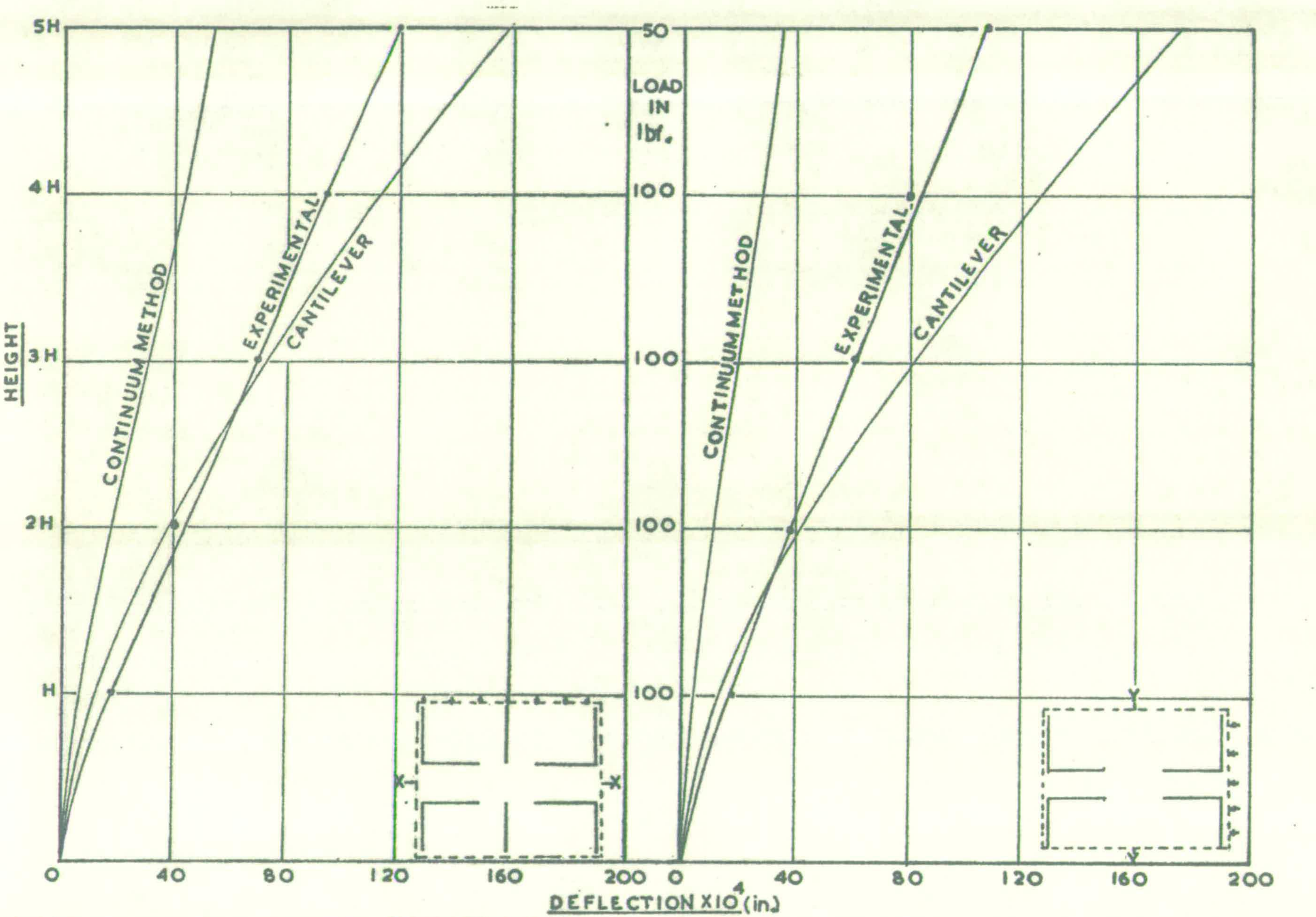


FIG. 7.13 COMPARISION BETWEEN DEFLECTIONS OBTAINED ANALYTICALY AND EXPIRIMENTAL RESULTS IN SECOND LOADING STAGE

$$\Delta_1 = \frac{Vh^3}{12EI} + \frac{1.2\ Vh}{AG} \qquad \text{(xxiii)}$$

Total deflection will be:-

$$\delta = \Delta_1 + \Delta_2 + \Delta_3 + \Delta_4 + \Delta_5 \qquad \text{(xxiv) (Fig.7.10)}$$

Now, following Soane[59], the entire flange was assumed acting with the corresponding wall. The deflections at each floor slab level were calculated and plotted in fig. 7.12 assuming the value of $G = \frac{E}{2.2}$ from beam test (P. 78). This method appeared very promising, and nature of the curve compared favourably with the test results described in this chapter as well as from Essex University model (Fig. 7.11, 7.12 and 7.14).

The method was further explored, though it is "a posteriori" approach, based on experimental results which gives a remarkable result. From single storey tests Chapter 6 (Table 6.2), it was clear that precompression increases the modulus of rigidity and in the multi-storey structure, the value of precompression varies from one storey to another. While in the above calculation it was assumed constant and taken from a small beam, which may be completely different in the structure. In the prediction of behaviour of the structure errors more than 50% can be introduced in the rigidity depending only on the workmanship and other factors. Hence, the results of preliminary tests - in x-x direction only, as shown in Table 7.2, were examined very carefully and values of G were calculated for each storey separately from the suggested formula (xxiii). From table 7.2 the value of G is 18.5×10^4 with a precompression of 50 lbf/in^2 against 22.1×10^4 with the precompression 55 lbf/in^2.

in/

in single storey test (Table 6.2), which is quite reasonable.

In the calculation of G, the assumption was made that no flange action takes place in loading from the direction of X-X due to the construction, which will be explained later on. In case of loading from the Y-Y direction only 1/6th of the cross-wall was assumed to act integrally with shearwalls, this is the flange width recommended for the design of L beam in reinforced concrete. Now the deflections at each storey were calculated from:-

$$\Delta_1 = \frac{(v_1 + v_2 \;\ldots\ldots + v_5)h^3}{12\, E_1\, I_1} + \frac{1.2\,(v_1 + v_2 \;\ldots\ldots\; v_5)h}{AG_1} \qquad \text{(xxv)}$$

where $E_1 = 2.2\, G_1$

$$\Delta_2 \quad \frac{(v_2 + v_3 \;\ldots\ldots + v_5)h^3}{12\, E_2\, I_2} + 1.2\,\frac{(v_2 + v_3 \;\ldots\ldots + v_5)h}{A_2\; G_2} \qquad \text{(xxvi)}$$

Total deflection at second floor level will be:-

$$\delta_2 = \Delta_1 + \Delta_2 \qquad \text{(xxvii)}$$

$$\delta = \Delta_1 + \Delta_2 + \Delta_3 + \Delta_4 + \Delta_5 \quad \ldots\ldots\ldots \text{(xxviii)}$$

The deflections obtained in X X and Y Y direction are shown in Fig. 7.11 and 7.12. The value was G calculated in x-x direction was assumed to hold good in y-y direction as well.

7.10 Discussion of the Results

The result of the test is compared with the code of practice and the factor of safety works out to be 4.6, with a design wind speed of 75 m.p.h./

75 m.p.h. The ultimate shear stress was 52.5 lbf/in^2 and when compared to the suggestion in section 6.6 the factor of safety is 2 and overall factor of safety is 2.6 as per code (Chapter V). This is not alarming, as live load stresses were completely ignored.

The ultimate shear stress of 52.5 lbf/in^2 is somewhat lower than predicted from the couplet formula (Fig. 6.6), but is within one standard deviation (68.5% confidence limit). As pointed out in Section 5, there is wide scatter in the value of initial bond shear and considering that the value of ultimate shear stress is not far out. In the couplet formula the shear strength is given by:-

$$\text{Vult} = V_{bo} + f\,\sigma y$$

$$= V_{bo} + .74 \times \sigma_y$$

By putting the value of Vult = 52.5 lbf/in^2 σy = 50 lbs/in^2, the value of V_{bo} = 15.5 lbf/in^2. It appears that the initial bond shear strength was lower which have led to the premature failure of the structure. There might be another possibility, if we look at the failure of the structure. Apart from one shear wall on the ground floor, all failed exhibiting cracks in the wall. This particular one (Plate 7.5) failed at the interface of the joint between slab and wall, which suggests that the joint was not perfect and shear was transferred only due to friction and thus full ultimate load was not reached at the time of failure of the structure.

The measured strains were very small and indeed below acceptable limits of accuracy; no conclusion can therefore be drawn from them.

There/

There is not much difference in the values of deflection at roof level in x-x and y-y direction Fig. 7.4 at lower loads which leads to the conclusion that perhaps there was no flange action while loading in the plane of cros-walls. This is understandable, as the cross-walls and shear-walls were joined throughout with mortar joint, which might permit deflection due to joint distortion without the participation of shear-wall as a flange at lower load. There is, however, marked difference at higher loads and perhaps the shear wall start acting as flange. Loading in the plane of the shear-walls, the flange action is evident as the shear wall is butting against the cross-walls (Fig. 7.1).

The deflections measured at different floor levels of the structure are compared in Fig. 7.11, 7.12 and 7.13 with the analytical values which represent the behaviour of idealised structures. The deflection profiles obtained experimentally from both directions are entirely different from existing idealised methods of analysis. The values, were, however, in between the cantilever and other approaches. As the load was increased from one stage to another, the deflection in ground floor and 1st floor became even larger than cantilever approach and almost linear. This points out that the existing theories do not represent the actual behaviour of brick shear-walls. In this particular structure the joint between slab and wall cannot be expected to transfer the full bending moment, hence each individual storey was considered separately. The entire width of cross-wall was taken acting as flange of shear wall. The result of this assumption is shown in/

in Fig. 7.12. Apart from the values, the profile of the curve is similar in form to the experimental curve. Having found this the profile was compared with those obtained by Soane[59] in the test of Essex University model made of homogeneous elastic material with rigid connection. There is considerable similarity in the nature of the curve (Fig.7.14). There was 74% difference between the experimental and predicted value of deflection which was attributed to foundation rotation. If this is taken as correct then the corrected profile should have the same form although the values may be somewhat different; but it is not. Now examining Fig. 7.12, the curves obtained by the continuum method and the one suggested in Section 7.9 intersect each other in the same pattern as that of Fig. 7.12 and 7.14 (theoretical and corrected profile). However, only 7% difference was found between theoretical and experimental result in case of mirror-image model, which was just a long narrow cantilever joined with beams. It appears that the deflection profile of the Essex University[59] building was not exactly represented by the mathematical model, and that the deflection was largely due to shear as in a deep beam.

Another variable parameter, which affects the rigidity of the shear wall is the integral action of the cross walls. As the structure was quite stiff, no relationship could be established by measuring strains. However, the theoretical and experimental results 7.11 and 7.12 are in good agreement with the assumption made in section 7.9. The reduction in the flange width will not close the gap between the experimental/

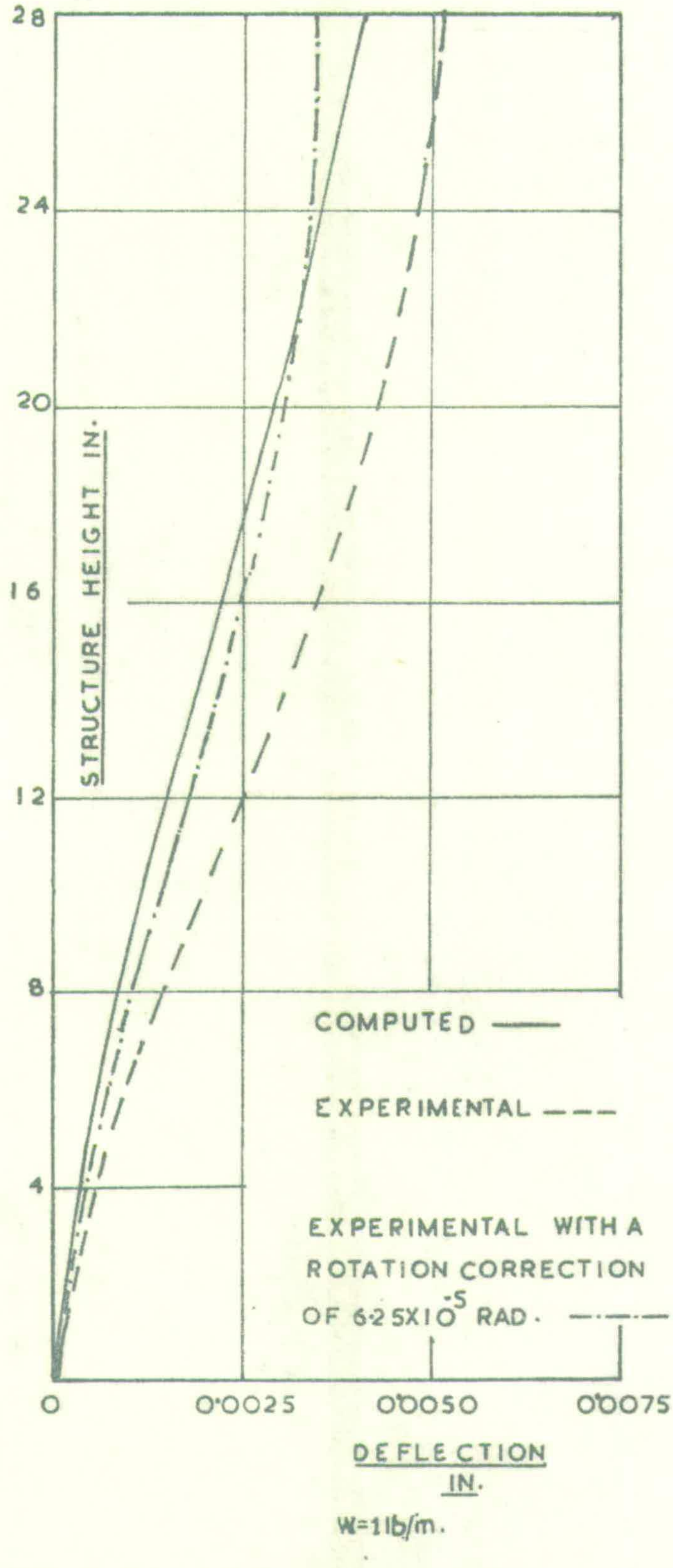


FIG. 7.14 DEFLECTIONS FOR ESSEX BLOCK

experimental and the theoretical idealised structure. The deflection, assuming it as a cantilever, will increase many times as the moment of inertia is reduced and by other two methods not very much effect will result, as the centre of gravity of the walls will shift further away from the assumed point of inflection in the lamine. Further intensive research on relatively flexible shear walls of large panel type is necessary to establish the flange action and to study the properties of the joints between the wall and slab as regards to the transfer of bending moment, before any exact solution could be put forward for the design of multi-storey brick cross-wall structures.

The rigidity of the structure is assumed as the load per unit deflection at roof level and from Fig. 7.15 the relationship between rigidity and horizontal load is non-linear.

Fig. 7.5, the relationship between deflections at different floor level and horizontal load is also non-linear, which confirms the results of single storey structure (Section 6.6).

As in the single storey structure some of the bricks failed in tension though the tensile stress was much below their tensile strength. This has already been explained in Section 6.6.

From Fig. 7.4, it appears that the deflection is largely due to shear/

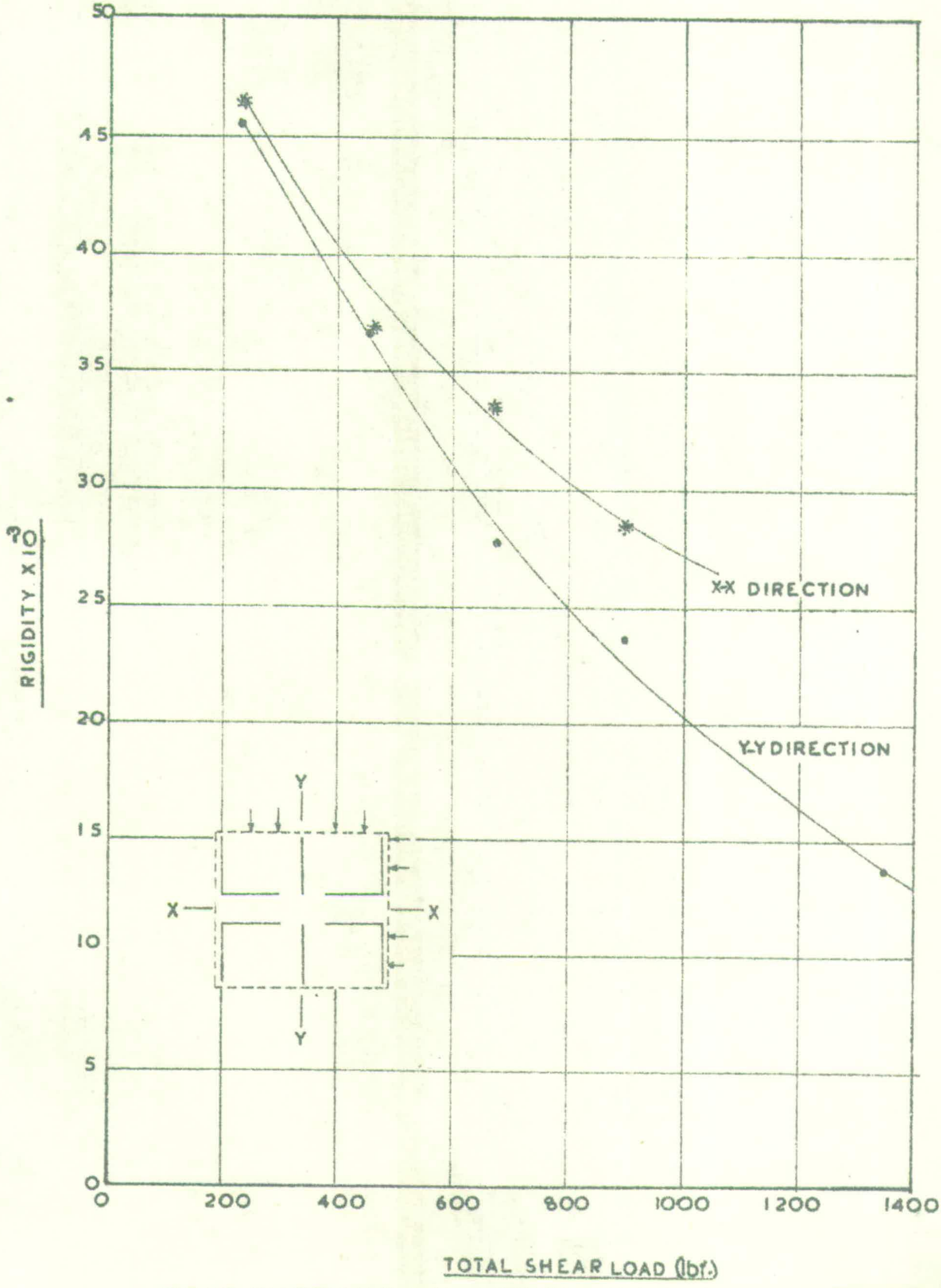


FIG.NO-7.15 RELATIONSHIP BETWEEN SHEAR LOAD AND RIGIDITY.

From Fig. 7.4, it appears that the deflection is largely due to shear and not due to bending and is reverse of that in skeleton framing. This has also been reported by Davison, Fisher and Monk[21].

7.11 Conclusion

1. The failure of a multi-storey shear wall structure with openings will occur in the first storey, assuming uniform thickness of walls and same bricks throughout. It will be due to the breakdown of the bond at the brick/mortar interface, leading to diagonal cracks stepping through the vertical and horizontal mortar joints. In some cases it may fail at the interface of the joint between slab and wall.

2. The load factor of 4.6 based on C.P.3, Chapter V, Loading and C.P.111 (1964) is quite reasonable in case of exposure (D). As compared to the suggestion in section 6.6, the overall factor of safety of 2.6 in case of exposure D is quite adequate taking into consideration that there were no live load stresses.

3. Existing analytical solutions do not apply to a multi-storey brickwork structure. Due to its construction large panel buildings or multi-storey brick-cross wall structures behave as a series of deep beams, where shear deflection is predominent rather than bending deflection.

4. The behaviour of multi-storey complete cross-wall structure could be predicted approximately on the basis of single storey theory considering shear deflection only. The deflection curve in a multi-storey structure of this type will be reverse of that for a skeleton framework.

5./

5. The relationships between deflections, rigidity and the shear loads are non-linear (Fig. 7.15, 7.5). The rigidity decreases with the increase of horizontal loading.

6. The rigidity in the multi-storey structure varies over the height of the building with the variation of pre-compression (see also section 6.7).

TABLE 7.2

Showing the relationship between deflection with position of Horizontal Loading.

Position of Load	Load in lbs.	Deflection x 10^{-4}					Value of $G \times 10^{-4}$
		1st Floor	2nd Floor	3rd Floor	4th Floor	5th Floor	
1st Floor	300	5.5	8.5	13.0	17.0	19.0	18.5
2nd Floor	120	3.2	8.3	14.3	19.0	21.3	10.4
3rd Floor	50	0	3.0	7.5	10.0	12.0	5.38
4th Floor	50	1.0	6.0	10.0	18.0	21.0	3.0
5th Floor	25	2	6	12	21	28.0	1.66

CHAPTER 8

General Conclusion

8.1 Introduction:

The conclusions arrived at as a result of the investigations are summarised in this chapter.

8.1.2 Load-Bearing Capacity of bonded walls

The test results on one-sixth scale model brick walls equivalent to 9-in. full scale indicate that there is no significant difference in the load-carrying capacity of brickwork in different bonds. The stretcher bond with ties can take 10% more load than without ties. These tests, simulating the actual end conditions of the wall in a building, indicate very high load factor on the basis of C.P.111 (1964). The tensile strength of the bricks were also high and as the usual mode of failure of wall in compression is due to splitting, it appears reasonable that the code should be revised and the allowable stresses should be based on tensile strength. This is necessary, because there appears to be no definite relationship between the tensile strength of brick and the compressive strength and will help in comparing the test results of different laboratories. The compressive strength is quite arbitrary and is defined by testing the bricks between plywood sheets. Results may vary if test conditions are changed. However, a reliable method for determining the tensile strength of brick has yet to be found. The usual method of comparison between brickwork strength: brick strength is out of date, being unrealistic and/

and cannot be relied upon for comparing the test results of different laboratories. The variation of the secant modulus of elasticity with compressive stress was found linear with negative slope.

8.13 Bond Tension, Bond Shear and Shear Strength of Couplets Subjected to Precompression.

The bond strength of model brickwork with 1:3 mortar varies considerably with the moisture content of brick and for maximum bond there is an optimum value of moisture content for a particular mortar. No definite recommendation could be made for the design of brickwork with regard to mortar adhesion and the development of tensile strength perpendicular to the mortar joint, but it should be left to the judgement of the designer depending on the site condition. However, code may allow some tension depending on the shear strength of the brickwork according to the formula suggested in Chapter 5 namely, $V_b = 8.8\ f_t^{0.5}$ as a guide (Fig. 5.5. and 5.6). The bond tension and bond shear are independant of the load placed on them during the curing period.

The shear strength increased with the increase of precompression.

8.14 Shear Test on Single Storey Structure with Opening

In the brick couplet test the shear was applied to the mortar joint and no shear developed in bricks and to this extent the couplets were not a true representation of the actual structure. In an actual structure the presence of shear will give rise to diagonal tension and compression in the masonry. Hence, in the tests the model structures were subjected to precompression first before the shear load was applied. The test results/

results indicate that the failure of brickwork will be of two distinct types:

1. Shear failure at the brick-mortar interface, governed by initial bond shear and frictional resistance between brick and mortar due to precompression.

2. By cracking through brick and mortar, governed by maximum tensile stress or strain. A theoretical formula has been suggested based on the experimental results which give results comparing favourably the observed ultimate strength of the structure. The results were compared with C.P. 111 (1964) and it was found that the specified working stresses in shear are not realistic and a suggestion has been made for the revision of the relevant clause in the code. The test results are in good agreement with the results on full scale couplets and individual walls; which will make it possible in future to study the behaviour and ultimate strength of full-scale brickwork for given type of brick and mortar by model test.

Precompression increases the shear strength, rigidity and shearing modulus. The shear modulus and rigidity decrease with the increase of horizontal load and the relationship is non-linear.

8.15 Multi-storey Cross-wall-type Structure

Under wind loading the failure of multi-storey brick cross-wall structure with uniform wall thickness and similar bricks throughout will take place in the lowermost storey by cracking through vertical and horizontal/

horizontal mortar joint or by failure of joint at the interface of wall and slab. The load factor when compared to code of practice Chapter V "Loading" for 5 storey high crosswall structure was 4.6 on the basis of C.P.111 (1964) and 2.6 on the basis of the suggested revision (Section 6.6). As these load factors are calculated without considering live load stresses, they would appear to be adequate. The usual analytical solution based on the idealised structure does not hold good for the brick crosswall structure tested where deflection is largely due to shear.

A simple approach based on a discontinuous stack of single storey structures is in good agreement with the experimental results. The value of G varies from one storey to another due to the variation in the value of precompression. The calculation of the rigidity of such a structure should be based on this simple approach till further analytical and experimental work is done. As in the case of a single storey structure the shear modulus and rigidity of the structure decreases with the increase of horizontal load and the relationship between them is non linear.

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